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JAN 77 L I KNAB, R C MOODY, W W SANDERS

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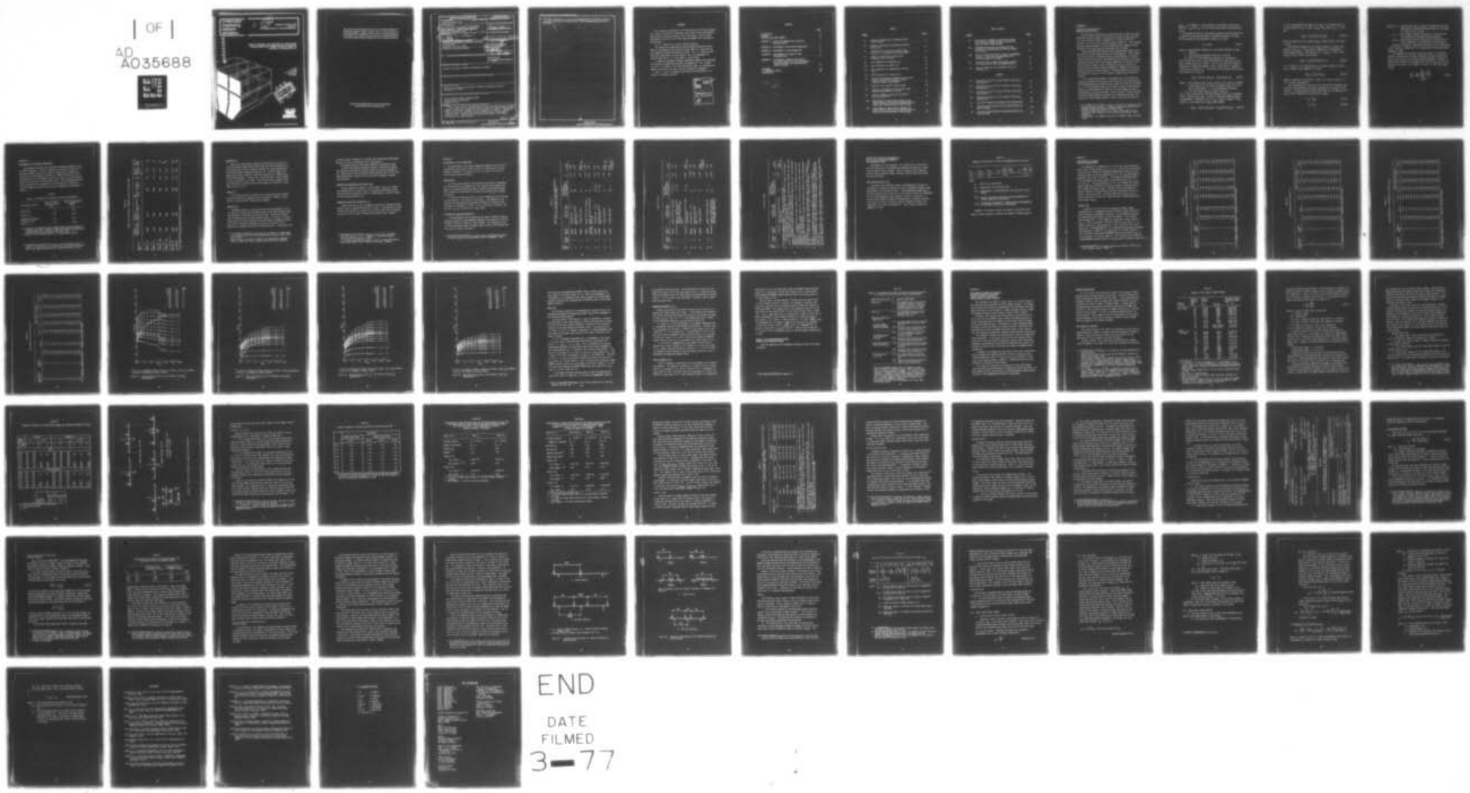

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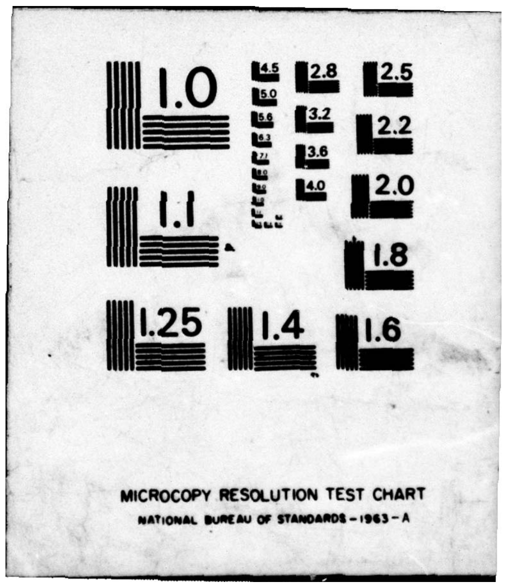
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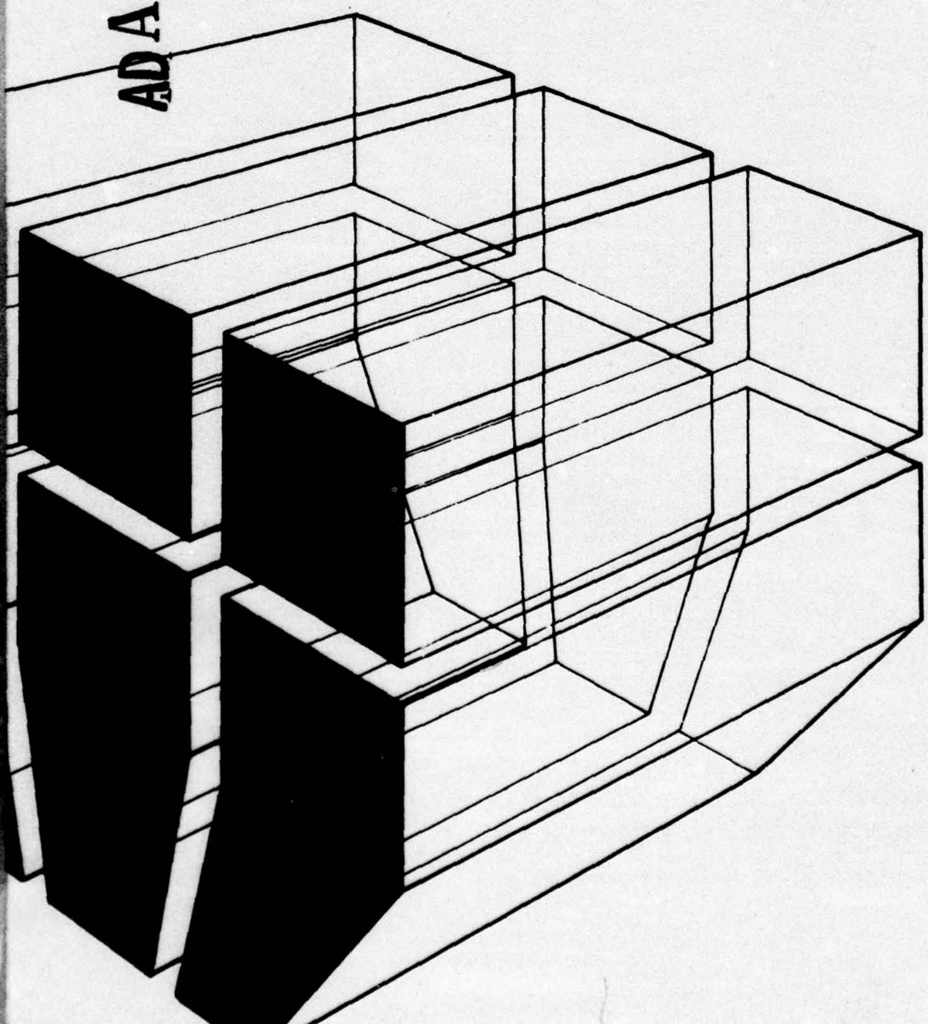
AFCS Design Parameters for T/O Material Applications

DESIGN CRITERIA FOR THEATER OF OPERATIONS GLUED-LAMINATED TIMBER HIGHWAY BRIDGES
VOLUME II: APPENDICES A-E



by
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This volume contains the development of and justification for the design criteria, procedures, and material specifications for glued-laminated timber Theater of Operations highway bridges recommended in Volume I. Appendices A through D describe the method and assumptions used in developing the design criteria. Appendix E describes the development of the moment distribution and shear formulas.

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These appendices do not provide recommendations for design criteria, procedures, and material specifications; the recommendations are given in Volume I.

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FOREWORD

This study was conducted for the Directorate of Facilities Engineering, Office of the Chief of Engineers (OCE) under Project 4A763734DT34, "Development of Engineer Support to the Field Army"; Task 04, "Base Development"; Work Unit 002, "AFCS Design Parameters for T/O Material Applications."

Mr. R. H. Barnard was the OCE Technical Monitor.

The research was conducted by the Construction Materials Branch (MSC) of the Materials and Science Division (MS), U.S. Army Construction Engineering Research Laboratory (CERL). Dr. L. I. Knab was CERL Principal Investigator for the project. Mr. P. A. Howdyshell is Chief of MSC and Dr. G. R. Williamson is Chief of MS.

R. Moody of the Forest Products Laboratory, Madison, WI, developed the resistance assumptions presented in Appendix B. W. W. Sanders, Jr. and H. A. Elleby of Iowa State University, Ames, IA, developed the moment distribution and shear formulas presented in Appendix E.

COL J. E. Hays is Commander and Director of CERL and Dr. L. R. Shaffer is Technical Director.

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APPENDIX A:

RELIABILITY METHOD USED TO DEVELOP DESIGN CRITERIA

The reliability method used to develop design criteria for glued-laminated stringer bridges is essentially the same as that detailed for steel stringer bridges in Appendix A of *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II.[1] This appendix describes the development of the one modification introduced, which accounts for the load duration characteristics of timber.

Tests have shown that wood can carry substantially greater maximum loads for short durations than for long durations.[2] This property involves the wood's resistance as well as the duration of the load effect* acting on the wood. The expression for the ratio of the mean resistance to mean load effect (R_m/Q_m) appearing in the safety index beta expression given in Eq A2 of *Design Criteria for Theater of Operations Steel Highway Bridges* (Appendix A, Volume II) must be adjusted to account for this property. Although either R_m or Q_m could be adjusted to account for this property, adjusting Q_m is more convenient.

In the approximate method of adjustment used, the 10-year continuous or cumulative load duration was used as a base load duration; the corresponding mean ultimate stress, based on 10 years, was called f_m. If a load duration other than 10 years is used, a mean ultimate stress f_{mt} corresponding to t years can be used; however, using f_m instead of f_{mt} is more convenient. Use of f_m with a t year load duration can be approximately accounted for by adjusting the mean load effect by a load duration factor T, which is equal to the ratio of

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1 L. I. Knab, et al., *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II, Technical Report M-195 (Construction Engineering Research Laboratory [CERL], 1977).

2 *National Design Specification for Stress Grade Lumber and Its Fastenings (NDS)* (National Forest Products Association, 1973), Appendix H.

* A load effect is a member force such as a moment, shear, or axial force.

f_m/f_{mt}. For example, if a load duration of less than 10 years were used ($f_m < f_{mt}$) *with* f_m, *the load effect would need to be decreased by* f_m/f_{mt}.

To demonstrate how the load duration property can be accounted for in terms of the reliability analysis equations, the mean live load effect adjusted for load duration, called L_m, is used:

$$L_m = T_L L'_m \quad \text{[Eq A1]}$$

where L_m = mean maximum lifetime live load effect adjusted for load duration

T_L = load duration adjustment factor for live load

L'_m = mean live load effect not adjusted for load duration.

Eq A1 is used only when a t year load duration is used with f_m, the mean ultimate stress for 10 years. Eq A1 is not used for 10 years of *continuous or cumulative loading, since in that case* T_L is taken as unity and L_m is equal to L'_m. In the case of the 10-year load duration, the expression for R_m/Q_m is:

$$R_m/Q_m = [f_m/(\gamma F_a)](L_n/L_m) = [f_m/(\gamma F_a)](L_n/L'_m) \quad \text{[Eq A2]}$$

where γ = the allowable stress factor, which is the ratio of allowable stress used to F_a, the 10-year allowable stress; for a 10-year load duration, γ = 1.0 (for permanent structures)

L_n = live load effect based on nominal code load.

For a t year load duration, the mean ultimate stress f_{mt} (corresponding to t years) can be used. In the live load case being discussed, no adjustment is necessary for R_m if f_{mt} is used; T_L equals 1 and L_m equals L'_m. The expression for R_m/Q_m then becomes

$$R_m/Q_m = [f_{mt}/(\gamma F_a)](L_n/L_m) = [f_{mt}/(\gamma F_a)](L_n/L'_m) \quad \text{[Eq A3]}$$

If, for a load duration not equal to 10 years, the 10-year stress f_m is used instead of f_{mt}, T_L equals f_m/f_{mt} and L_m equals $T_L L'_m$. In this case

$$R_m/Q_m = [f_m/(YF_a)][L_n/(T_L L'_m)] \qquad \text{[Eq A4]}$$

Note as a check that if T_L equals f_m/f_{mt} is used in Eq A4, the result is Eq A3.

The following example for live load is based on a 2-month load duration, with $f_{mt} = 1.15f_m$ (based on Appendix H of the National Design Specification). This results in $T_L = f_m/f_{mt} = 1/1.15 = 0.87$. Using T_L equal to 0.87 in Eq A4 gives

$$R_m/Q_m = [f_m/(YF_a)][L_n/0.87\ L'_m)] \qquad \text{[Eq A5]}$$

If Y is taken as 1.15 (representing a 15 percent increase in the 10-year allowable stress--Appendix H of NDS),

$$R_m/Q_m = (f_m/F_a)(L_n/L'_m) \qquad \text{[Eq A6]}$$

which is equivalent to Eq A2 with Y equal to 1.0 and T_L equal to 1.0 for a 10-year load duration.

For a combination of loads with each load having a different load duration, the load effect can again be adjusted by a duration of load factor T. The mean maximum lifetime load effect for the dead and live load effects, adjusted for load duration, becomes

$$D_m = T_D D'_m \qquad \text{[Eq A7]}$$

$$L_m = T_L L'_m \qquad \text{[Eq A8]}$$

where T_D, T_L = load duration factors (f_m/f_{mt}) for dead and live load respectively, used when the load duration is not equal to 10 years and f_m is used

D_m, L_m = mean maximum lifetime load effects for dead and live load respectively, adjusted for load duration

D'_m, L'_m = mean maximum lifetime load effects for dead and live loads respectively, not adjusted for load duration.

Although this method is approximate for more than one load effect, it does provide a rational basis for accounting for the load duration property of timber.

The framework for the wood reliability analysis is thus represented by the equations previously developed in *Design Criteria for Theater of Operations Steel Highway Bridges* (Appendix A, Volume II) using, where appropriate, the definitions of D_m and L_m given by Eq A7 and A8 of this report. For example, for the D + L load case, R_m/Q_m becomes

$$\frac{R_m}{Q_m} = \left(\frac{f_m}{\gamma F_a}\right)\left(\frac{\frac{D_n}{T_L L'_m} + \frac{L_n}{T_L L'_m}}{\frac{D_m}{T_L L'_m} + 1}\right) \qquad \text{[Eq A9]}$$

APPENDIX B:

DEVELOPMENT OF RESISTANCE ASSUMPTIONS

This appendix presents the resistance assumptions needed for the reliability analysis. Table B1 summarizes the values of the ratio of the mean ultimate stress for 10 years of continuous or cumulative loading (f_m) to the American Institute of Timber Construction (AITC) specification 117-74[3] allowable stress for 10 years (F_a) and the coefficient of variation of the resistance (V_R). The following discussion briefly describes the development of the information in Table B1; Table B2 summarizes the actual data used to develop the assumptions in Table B1.

Table B1

Summary of Probabilistic Assumptions Used for Resistance

Resistance Property F	Ratio of Mean to Allowable Stress f_m/F_a	COV* of Resistance Property V_R
Bending, F_b	1.73	0.14
Tension, F_t	1.51	0.23
Compression parallel to grain, $F_{c\|\|}$**	1.74	0.12
Shear, F_v	1.73	0.14

* The COV of resistance (V_R) was assumed equal to COV of material strength (V_M) given in Table B2, since V_M includes the uncertainty in fabrication (V_F) and strength prediction (V_P).

** Refers to fully braced column (i.e., column cannot buckle).

[3] *Standard Specification for Structural Glued Laminated Timber of Douglas Fir, Western Larch, Southern Pine, and California Redwood*, AITC 117-74 (American Institute of Timber Construction, 1974).

Table B2

Summary of Resistance Information--Based on Actual Data

Resistance Type	5-Minute Mean Ultimate Stress, f'_m (psi)	10-Year Adjusted* Mean Ultimate Stress, f_m (psi)	NDS 10-Year Allowable Stress, F_a (psi)	$\frac{f_m}{F_a}$	COV of Material Strength V_M	No. of Speci-mens
$F_{c\|\|}$, L3 Douglas fir	4090	2710	1500	1.81	0.12	26
$F_{c\|\|}$, L2 Douglas fir	4710	3120	1800	1.73	0.12	25
$F_{c\|\|}$, No. 2 Southern pine	4600	3050	1900	1.61	0.12	25
$F_{c\|\|}$, No. 3 Southern pine	3840	2540	1400	1.81	0.12	25
F_t, Tension, L3 Douglas fir	2940	1810	1200	1.51	0.23	180
F_b, Bending	6720	4145	2400	1.73	0.14	86

* $f_m = f'_m/1.51$ for $F_{c||}$ and $f'_m/1.62$ for F_t and F_b.

Bending (F_b)

Data on the actual mean strength and coefficient of variation (COV) for nominal 2400f glulam beams (Table B2) were obtained by combining results of several studies of Douglas-fir beams conducted by the Forest Products Laboratory, Madison, WI. A total sample of 86 beams from several different studies was adjusted to a common basis and grouped together.[4] Results were consistent with a similar number of tests conducted in the 1940s. Equivalent 10-year strength values were obtained by dividing 5-minute test values by a duration of load factor value of 1.62.

Shear (F_v)

As no extensive data on shear exist, using actual data in developing the resistance assumptions was not possible. Rather, factors similar to those used for bending were assumed.

Tension (F_t)

Equipment capable of testing multiple-ply glulam members in tension has only recently been developed; most of the data obtained using the equipment to date have not been published. These data indicate that the methods used to assign design stresses in tension in current specifications are not consistent with those used for bending strength. Data given in Table B2 represent the results of 180 tests of L3 Douglas-fir specimens with three or more laminations.[5] Additional data

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[4] R. C. Moody, *Glulam Beam Design Criteria*, Proposed U.S. Department of Agriculture Forest Service Research Paper (Forest Products Laboratory, 1976).

[5] John Peterson, *The Tensile Strength of L3 Douglas-Fir Laminated Members*, Unpublished Report (Oregon State University, undated).

on other grades of Douglas fir indicate that relationships developed based on L3 will be conservative for other grades.[6]

The same duration of load factor applied to bending was used.

It should be noted that design stresses in tension for glulam are presently (1976) being revised so that the factors developed in this report which relate to present specifications (AITC 117-74) will need modification if they are to be used with subsequent industry specifications.

Compression Perpendicular to Grain ($F_{c\perp}$)

Because moderate increases in the allowable stress for compression perpendicular to grain result in increased deformations rather than structural collapse in nearly all applications, a specific analysis was not performed for $F_{c\perp}$.

Compression Parallel to Grain ($F_{c||}$)

Unpublished data on two-ply members were used to evaluate current design levels in compression parallel to grain.[7] The duration of load factor was assumed to be slightly less in this factor than in bending; a value of 1.51 was used to relate 5-minute and 10-year loadings.

[6] John Peterson, *The Tensile Strength of One-, Two-, and Three-Lamination Members of 2 x 6 Douglas-Fir*, Unpublished Report (Forest Products Laboratory, 1975).

[7] R. C. Moody, *Compressive Strength of Two- vs. Single-Ply Members*, Unpublished Report (Forest Products Laboratory, undated).

APPENDIX C:

DEVELOPMENT OF LOAD ASSUMPTIONS

The development of the load assumptions needed for the reliability analysis used in this study was similar to that given for steel bridges in Appendix C of *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II.

Dead Load(D)

As in the steel analysis, the mean dead load effect (unadjusted for load duration (see Appendix A of this report) was assumed to be equal to the nominal dead load effect, based on code specifications; the coefficient of variation of the dead load effect V_D was assumed to be 0.06.

The dead load duration factor T_D varies depending on the lifetime of the structure. The values of T_D ($=f_m/f_{mt}$ - see Appendix A) are based on the relationship of strength to duration of load as given in Figure H1, Appendix H, of NDS. For 50- and 5-year lifetimes, the values of T_D are 1.04 and 0.97, respectively.

Live Vehicle Load and Payload (L)

The development of the assumptions for loads, allowable stresses, and L_n/L'_m values for both permanent (AASHTO)[8] and temporary (military) bridges is similar to that for steel bridges. The assumptions used for glued-laminated timber stringers (some of which are different than for steel) are given in Table C1.

[8] *Standard Specifications for Highway Bridges* (American Association of State Highway and Transportation Officials [AASHTO], 1973).

Table C1

Combinations of L_n/L_m' and γ Used in Beta Analyses for Glued-Laminated Bridge Members

Load Case No.	Bridge Type[a]	Specification	Crossing Type (Loading Used)	Resistance (Failure Mode) (See Appendix B)	γ^b	L_n/L_m'[c]
1	P	AASHTO	Overload (Art. 1.2.4)	F_b	1.50	1.25[d]
2	P	AASHTO	Overload with Permit (Art. 1.11.1)	F_b[e]	1.33	1.25[d]
3	P	AASHTO	Illegal Overload[f]	F_b	1.00	0.83 (1.25/1.50)
4	P	AASHTO	Normal (Civilian) Crossing	F_b	1.00	1.25[d]
5	T	Military (TM 5-312)	Normal and Caution Military[g] Crossings	F_b	≥1.10	1.25[h]
6	P	AASHTO	Overload (Art. 1.2.4)	F_v, $F_{c\|\|}$	1.50	1.00
7	P	AASHTO	Overload with Permit (Art. 1.11.1)[i]	F_v, $F_{c\|\|}$	1.33	1.00
8	P	AASHTO	Illegal Overload[f]	F_v	1.00	0.67 (1.00/1.50)
9	P	AASHTO	Illegal Overload[f]	$F_{c\|\|}$	1.00	0.80 (1.00/[(1.5 +1.0)/2])

Table C1 (cont)

Load Case No.	Bridge Type[a]	Specification	Crossing Type (Loading Used)	Resistance (Failure Mode) (See Appendix B)	γ[b]	L_n/L_m'[c]
10	P	AASHTO	Normal (Civilian) Crossing	F_v, $F_{c\|\|}$	1.00	1.00
11	T	Military[j]	Normal or Caution Military Crossing Used--Choose Whichever Produces Larger Force	F_v, $F_{c\|\|}$	≥1.05	1.00
12	T	Military[k]	Force Due to Normal Military Crossing Used	F_v, $F_{c\|\|}$	≥0.85	0.80 (1.00/1.25)
13	P	AASHTO	Overload (Art. 1.2.4)	F_t	1.125[l]	1.00
14	P	AASHTO	Overload with Permit (Art. 1.11.1)	F_t	1.00[l]	1.00
15	P	AASHTO	Illegal Overload[f]	F_t	0.75[l]	0.80 (1.00/[1.5 +1.0)/2])
16	P	AASHTO	Normal (Civilian) Crossing	F_t	0.75[l]	1.00
17	T	Military[j]	Normal or Caution Military Crossing Used--Choose Whichever Produces Larger Force	F_t	≥ 0.80[l]	1.00

Table C1 (cont)

Load Case No.	Bridge Type[a]	Specification	Crossing Type (Loading Used)	Resistance (Failure Mode) (See Appendix B)	Y^b	L_n/L'_m [c]
18	T	Military[k]	Force Due to Normal Military Crossing Used	F_t	≥0.65[l]	0.80 (1.00/1.25)

a P = permanent (AASHTO) bridge; T = temporary (military) bridge.

b Y = ratio of allowable stress used to the NDS allowable stress for 10 years of continuous or cumulative loading. (See footnote 1.)

c L_n/L'_m = ratio of nominal (code) live load effect to the mean maximum lifetime live load effect (unadjusted for load duration--see Appendix A).

d Average value based on typical glued-laminated stringer AASHTO bridge with a glued-laminated timber deck.

e Actually Y = 1.33 applies only to tensile stress, but Y = 1.33 was assumed for tension and compression in bending.

f Corresponds to a two-lane bridge loaded with a vehicle 50 percent over legal weight limit alongside a legal weight vehicle.

g Both normal and caution military crossings were assumed to produce the same maximum moment--see Appendix E.

h Average value based on a solid deck (glued-laminated timber or concrete)--see Appendix E.

i Article 1.11.1 does not cover shear, but case used for comparative purposes; Y = 1.33 assumed for $F_{c||}$.

j Assumes use of recommended procedure to determine shear force given in Appendix E.

k For shear, corresponds to using the recommended procedure in Appendix E, except replacing "1.25" in Table E8 with "1.00"; that is, only the normal crossing is used to determine the shear force.

l 0.75 of the NDS allowable stress used as permanent allowable stress in computing Y (see Tension discussion, Appendix B); 1.125 = 1.50 x 0.75; 1.00 = 1.33 x 0.75; 0.75 = 1.00 x 0.75.

Coefficient of Variation Assumptions for Live Load Effect for Permanent and Temporary Bridges

Development of the assumptions for coefficient of variation of the live load effect for permanent and temporary bridges was similar to that used for steel bridges. Table C2 presents the assumptions used for glued-laminated timber stringers (some of which are different than for steel).

Load Duration Factor (T_L)

Currently, the allowable stresses corresponding to 10 years of continuous or cumulative loading are used for permanent bridges, which have design lives of about 50 years. Since temporary bridges have 2- to 5-year design lives, using a live load duration factor T_L corresponding to 2 years appears to be reasonable. Hence a T_L value of 1.0 was used for permanent bridges (corresponding to a 10-year load duration), and a T_L value of 0.94 was used for temporary bridges (corresponding to a 2-year load duration). The T_L values are given in Appendix H of NDS.

Table C2

Summary of Coefficient of Variation Assumptions for Live Load

V_{L1}	V_{L2A}	V_{L2B}	$V_{L2} = \sqrt{V_{L2A}^2 + V_{L2B}^2}$	$V_L = \sqrt{V_{L1}^2 + V_{L2}^2}$
0.11	0.10	0.15	0.18	0.21

where

V_L = COV of live load effect

V_{L1} = uncertainty in the vehicle load

V_{L2} = uncertainty in transforming the live load into a load effect

V_{L2A} = actual observed uncertainty in the transformation of vehicle load into a load effect

V_{L2B} = uncertainty introduced in predicting the transformation of the vehicle load into a load effect.

Appendix C of *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II, explains the symbols in greater detail.

APPENDIX D:

DEVELOPMENT OF ALLOWABLE STRESS RECOMMENDATIONS

The information in Appendices A, B, and C was used to calculate safety index* (beta) values for F_b, F_v, $F_{c||}$, and F_t for the dead plus live load case. Tables D1 through D4 show the beta values for each resistance type for various combinations of Y and L_n/L'_m (developed in Appendix C and listed in Table C1). The values of the mean dead to live load effect ratio D_m/L_m shown in the tables are 0.0, 0.1, 0.3, 1.0, and 25.0. The D_m/L_m values of 0.1 to 1.0 represent the typical design range; the 0.0 and 25.0 values are given to show beta values for extreme load effect ratios. The beta values are graphically shown in Figures D1 through D4 for the four resistances and for selected Y and L_n/L'_m values. The load effect scale (D_m/L_m) of the figures consists of a series of linear portions to show trends over a wide range of D_m/L_m.

Bending (F_b)

A value of Y of 1.35 for F_b for temporary bridges appears reasonable and is recommended provided that the lateral load distribution formulas recommended in Appendix E and the moments caused by the normal military crossing are used for design. The following justification for Y = 1.35, based on a comparison of beta values at $D_m/L_m = 0.10$, is provided. The beta value for L_n/L'_m equal to 1.25 (case 5, Table C1), D_m/L_m equal to 0.1, and Y equal to 1.35 is 2.11 (Table D1). The beta value of 2.11 is 0.69 beta units above the AASHTO overload (case 1, Table C1) of L_n/L'_m equal to 1.25 and Y equal to 1.50; 0.20 beta units above the AASHTO overload with permit (case 2) of L_n/L'_m equal to 1.25 and Y equal to 1.33; 0.56 beta units above the illegal AASHTO overload (case 3) of L_n/L'_m equal to 0.83 and Y equal to 1.0;

* Also see Appendix A of *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II.

Table D1

Beta Comparison for Bending (F_b)

Load Case No. (Table C1)	Criteria Type*	γ	L_n/L_m'	Beta Value for D_m/L_m = 0.0	0.1	0.3	1.0	25.0
1	P	1.50	1.25	1.42	1.42	1.41	1.31	0.72
2	P	1.33	1.25	1.88	1.91	1.95	1.96	1.47
3	P	1.00	0.83	1.40	1.55	1.80	2.37	3.17
4	P	1.00	1.25	2.99	3.09	3.25	3.51	3.27
5	T	1.10	1.25	2.86	2.96	3.11	3.35	3.11
5	T	1.15	1.25	2.69	2.77	2.90	3.11	2.83
5	T	1.20	1.25	2.52	2.60	2.71	2.88	2.56
5	T	1.25	1.25	2.36	2.43	2.52	2.66	2.30
5	T	1.30	1.25	2.21	2.27	2.35	2.44	2.06
5	T	1.35	1.25	2.07	2.11	2.17	2.24	1.82
5	T	1.40	1.25	1.92	1.96	2.01	2.04	1.59
5	T	1.45	1.25	1.79	1.82	1.85	1.85	1.37
5	T	1.50	1.25	1.66	1.68	1.70	1.67	1.15

* P = permanent (AASHTO) bridge; T = temporary (military) bridge.

Table D2

Beta Comparison for Shear (F_v)

Load Case No. (Table C1)	Criteria Type*	Y	L_n/L_m'	Beta Values for D_m/L_m = 0.0	0.1	0.3	1.0	25.0
6	P	1.50	1.0	0.55	0.57	0.60	0.66	0.65
7	P	1.33	1.0	1.02	1.07	1.15	1.31	1.41
8	P	1.00	0.67	0.57	0.76	1.10	1.86	3.13
10	P	1.00	1.0	2.12	2.24	2.44	2.86	3.21
11	T	1.05	1.0	2.18	2.30	2.51	2.95	3.34
11	T	1.10	1.0	1.99	2.11	2.30	2.70	3.05
11	T	1.15	1.0	1.82	1.92	2.10	2.46	2.77
11	T	1.20	1.0	1.66	1.75	1.91	2.23	2.50
11	T	1.25	1.0	1.50	1.58	1.72	2.01	2.24
12	T	0.85	0.8	2.13	2.34	2.71	3.52	4.62
12	T	0.90	0.8	1.91	2.11	2.45	3.21	4.26
12	T	0.95	0.8	1.70	1.88	2.20	2.92	3.92
12	T	1.00	0.8	1.50	1.67	1.97	2.64	3.60
12	T	1.05	0.8	1.31	1.47	1.75	2.37	3.29

* P = permanent (AASHTO) bridge; T = temporary (military) bridge.

Table D3

Beta Comparison for Compression, Parallel ($F_{c||}$)

Load Case No. (Table C1)	Criteria Type*	Y	L_n/L_m'	Beta Values for D_m/L_m = 0.0	0.1	0.3	1.0	25.0
6	P	1.50	1.0	0.60	0.63	0.67	0.76	0.78
7	P	1.33	1.0	1.09	1.15	1.25	1.47	1.63
10	P	1.00	1.0	2.24	2.38	2.62	3.15	3.65
9	P	1.00	0.8	1.34	1.51	1.81	2.52	3.60
11	T	1.05	1.0	2.29	2.44	2.69	3.25	3.80
11	T	1.10	1.0	2.11	2.24	2.47	2.98	3.47
11	T	1.15	1.0	1.93	2.05	2.25	2.71	3.16
11	T	1.20	1.0	1.75	1.86	2.05	2.46	2.85
11	T	1.25	1.0	1.59	1.69	1.85	2.22	2.57
12	T	0.85	0.8	2.25	2.48	2.90	3.87	5.24
12	T	0.90	0.8	2.01	2.24	2.62	3.53	4.84
12	T	0.95	0.8	1.80	2.00	2.36	3.21	4.45
12	T	1.00	0.8	1.59	1.78	2.12	2.91	4.09
12	T	1.05	0.8	1.39	1.57	1.88	2.62	3.74

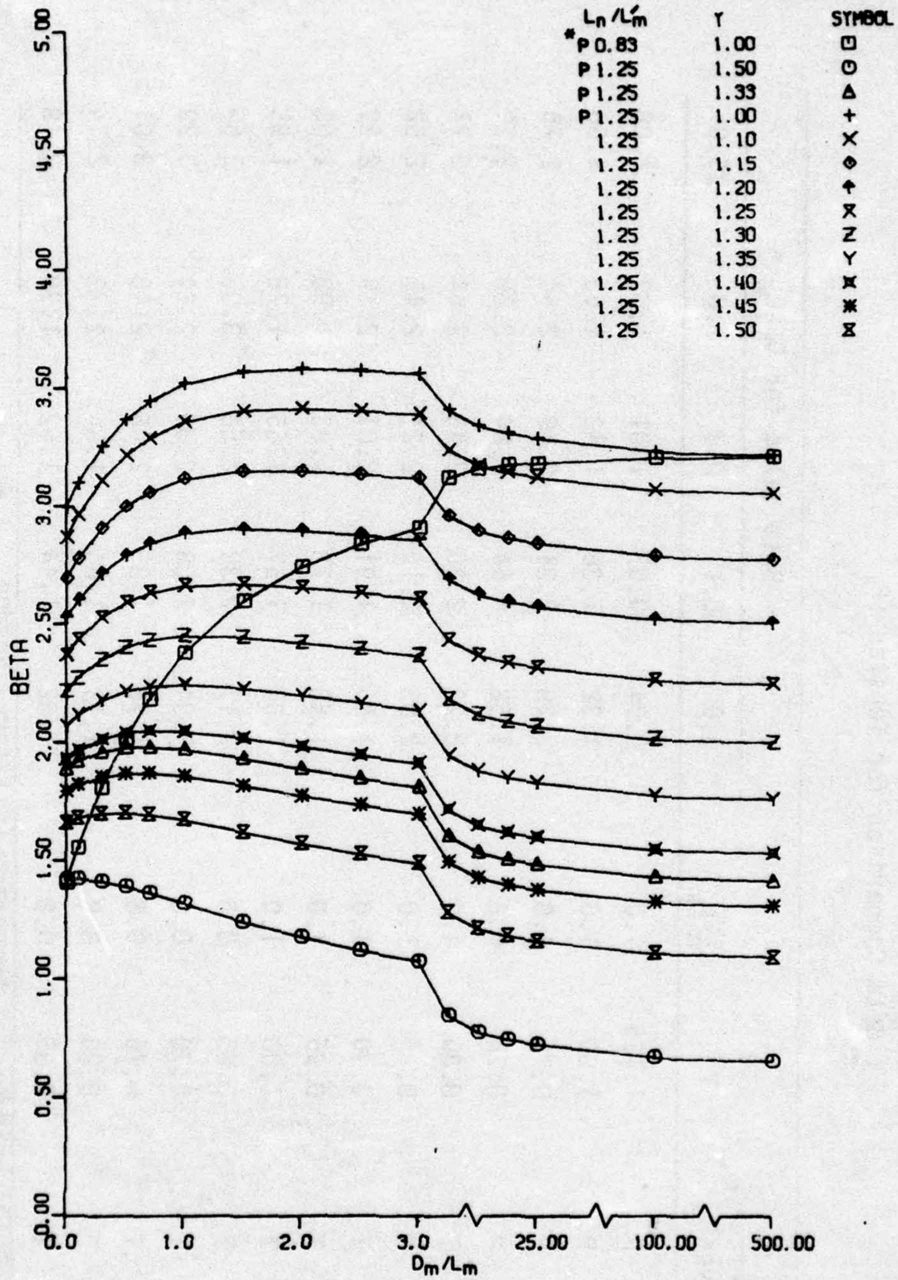
*P = permanent (AASHTO) bridge; T = temporary (military) bridge.

Table D4

Beta Comparison for Tension (F_t)

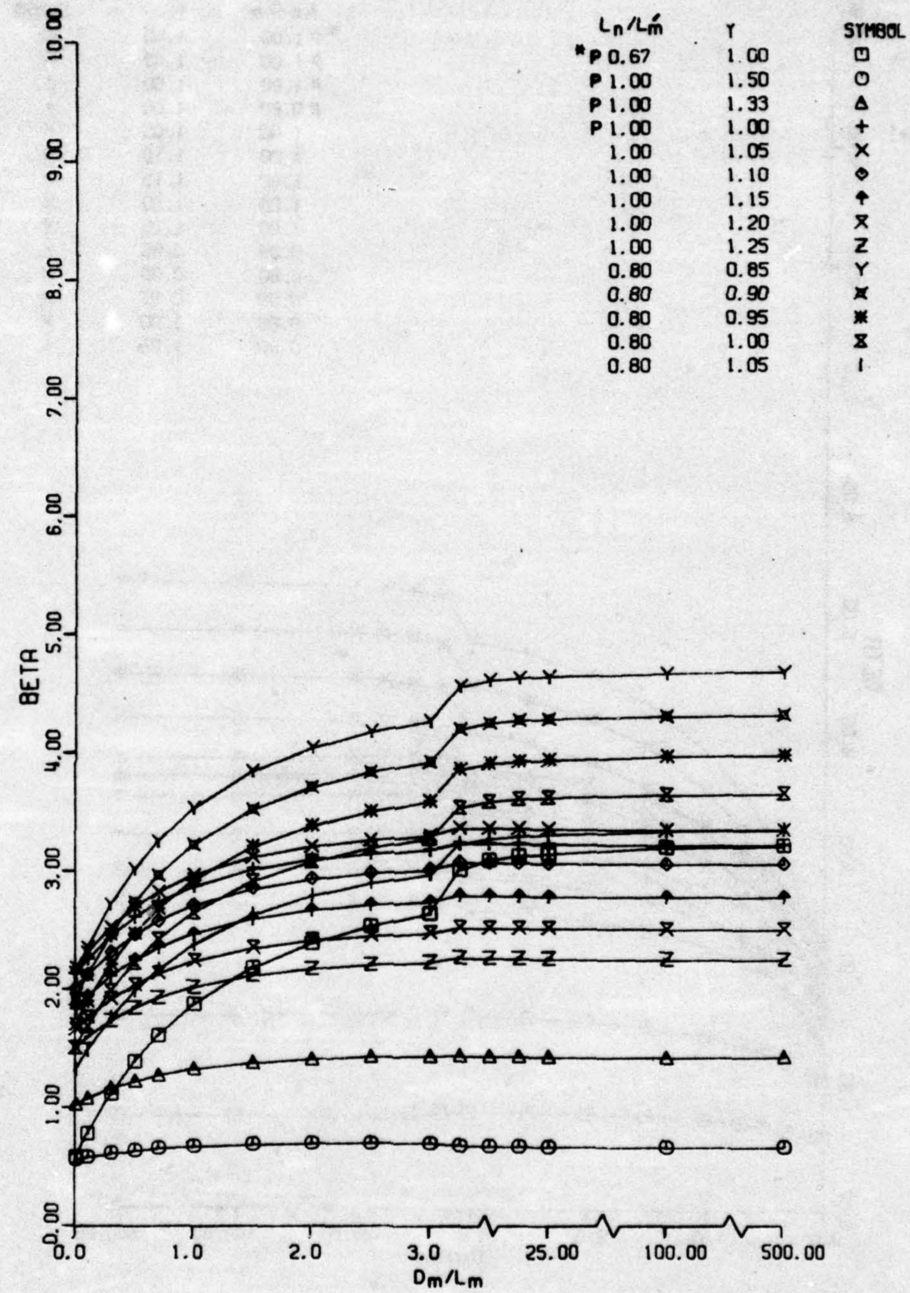
Load Case No. (Table C1)	Criteria Type*	Y	L_n/L_m'	Beta Value for D_m/L_m = 0.0	0.1	0.3	1.0	25.0
13	P	1.125	1.0	0.94	0.97	1.01	1.07	1.08
14	P	1.00	1.0	1.32	1.36	1.42	1.53	1.56
16	P	0.75	1.0	2.23	2.31	2.43	2.64	2.75
15	P	0.75	0.8	1.52	1.64	1.84	2.22	2.72
17	T	0.80	1.0	2.22	2.30	2.43	2.64	2.77
17	T	0.85	1.0	2.03	2.10	2.21	2.41	2.52
17	T	0.90	1.0	1.85	1.91	2.01	2.19	2.29
17	T	0.95	1.0	1.68	1.73	1.82	1.98	2.06
17	T	1.00	1.0	1.51	1.57	1.65	1.78	1.85
18	T	0.65	0.8	2.17	2.32	2.56	3.03	3.60
18	T	0.70	0.8	1.94	2.08	2.30	2.74	3.29
18	T	0.75	0.8	1.72	1.85	2.06	2.48	3.01
18	T	0.80	0.8	1.51	1.64	1.84	2.23	2.74
18	T	0.85	0.8	1.32	1.44	1.62	1.99	2.49

* P = permanent (AASHTO) bridge; T = temporary (military) bridge.



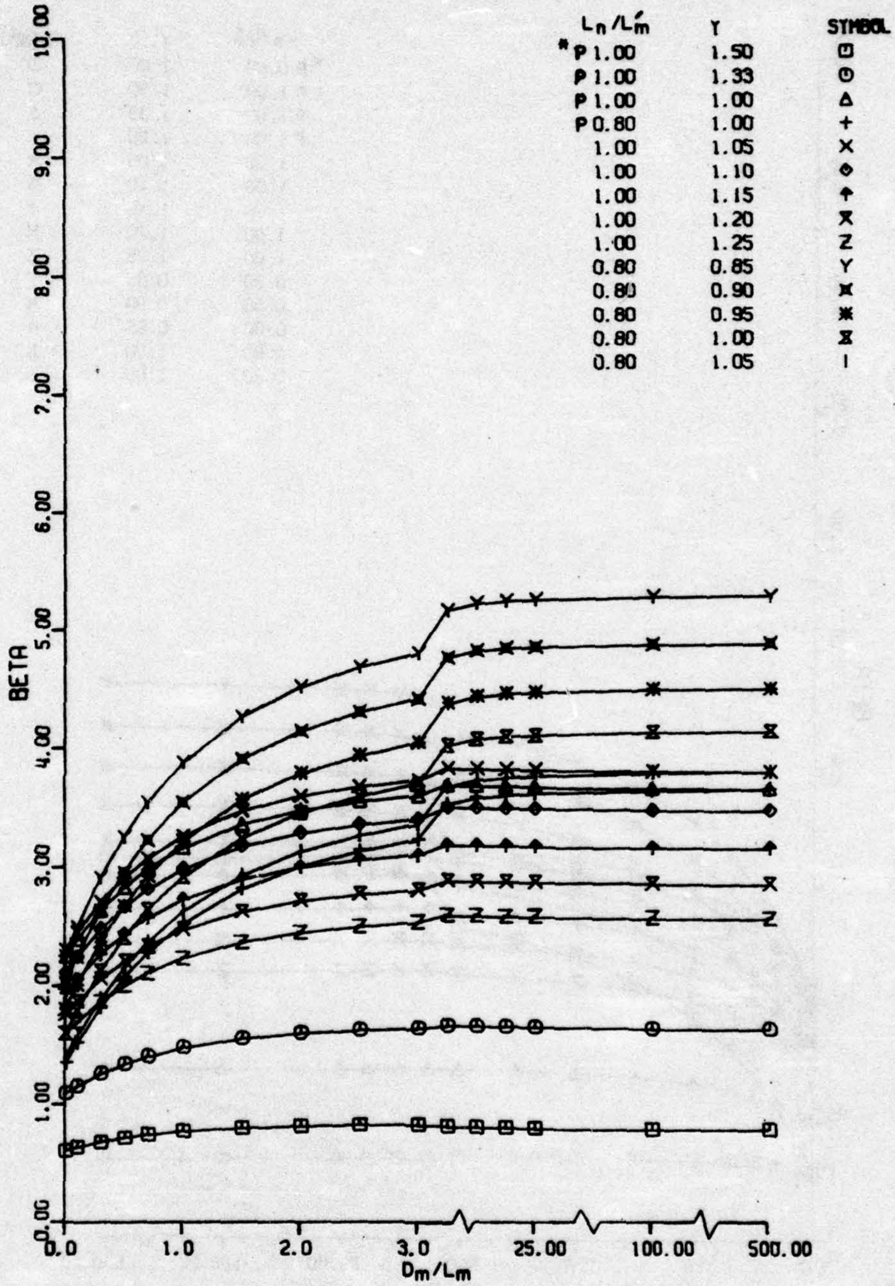
*P refers to permanent bridge criteria; all other Y and L_n/L'_m combinations are for temporary bridge criteria.

Figure D1. Beta versus D_m/L_m for F_b for various Y and L_n/L'_m combinations.



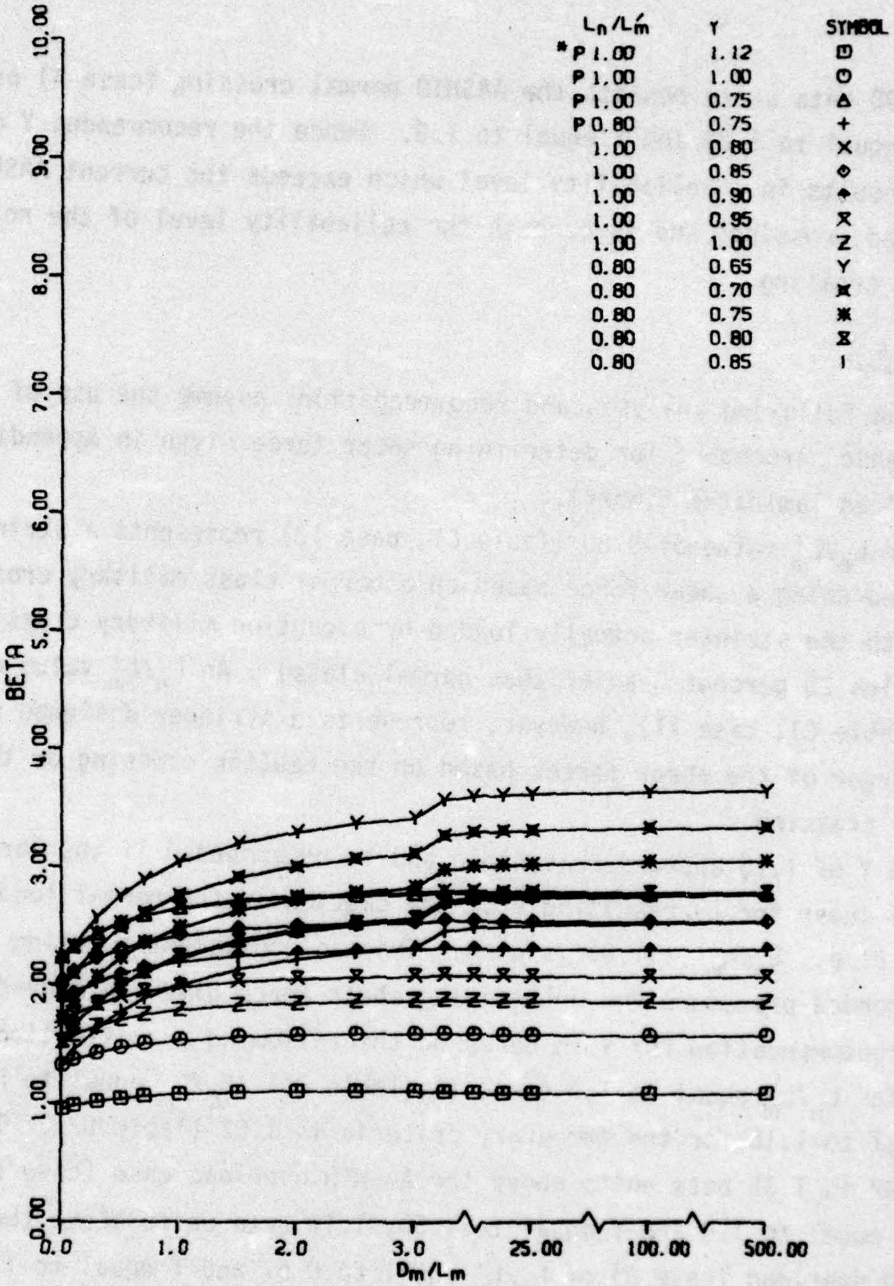
*P refers to permanent bridge criteria; all other Y and L_n/L_m' combinations are for temporary bridge criteria.

Figure D2. Beta versus D_m/L_m for F_v for various Y and L_n/L_m' combinations.



*P refers to permanent bridge criteria; all other Y and L_n/L'_m combinations are for temporary bridge criteria.

Figure D3. Beta versus D_m/L_m for $F_{c||}$ for various Y and L_n/L'_m combinations.



*P refers to permanent bridge criteria; all other Y and L_n/L'_m combinations are for temporary bridge criteria.

Figure D4. Beta versus D_m/L_m for F_t for various Y and L_n/L'_m combinations.

and 0.98 beta units beneath the AASHTO normal crossing (case 4) of L_n/L'_m equal to 1.25 and Y equal to 1.0. Hence the recommended Y of 1.35 results in a reliability level which exceeds the current AASHTO overload crossings and is beneath the reliability level of the normal AASHTO crossing.

Shear (F_v)

The following analyses and recommendations assume the use of the recommended procedure for determining shear force given in Appendix E (for glued-laminated timber).

An L_n/L'_m value of 0.80 (Table C1, case 12) represents a stringer designed using a shear force based on a normal class military crossing but with the stringer actually loaded by a caution military crossing (vehicles 25 percent heavier than normal class). An L_n/L'_m value of 1.0 (Table C1, case 11), however, represents a stringer designed using the larger of the shear forces based on the caution crossing or the normal crossing.

A Y of 1.15 appears reasonable and is recommended if the larger of the shear forces resulting from the caution or the normal load cases (i.e., L_n/L'_m = 1.0) is used. This is equivalent to using the recommended procedure for determining shear force given in Appendix E. This recommendation for Y is based on the following justification. The beta for L_n/L'_m equal to 1.0 (case 11, Table C1), D_m/L_m equal to 0.1, and Y equal to 1.15 for the temporary criteria is 1.92 (Table D2). The beta of 1.92 is 1.35 beta units above the AASHTO overload case (case 6) of L_n/L'_m equal to 1.0 and Y equal to 1.50; 1.16 beta units above the illegal AASHTO overload (case 8) of L_n/L'_m equal to 0.67 and Y equal to 1.0; and 0.32 beta units beneath the normal AASHTO load case (case 10) of $L_n L'_m$ equal to 1.0 and Y equal to 1.0.

If the normal military load case only is used* to determine the shear force (case 12, Table C1), a Y of 0.95 is recommended, since an

* This is equivalent to replacing the "1.25" coefficient in Table E8, Appendix E, with 1.00.

L_n/L'_m value of 0.80 can occur. The justification is similar to the shear for Y equal to 1.15 and L_n/L'_m equal to 1.0. It should be noted that the allowable stress for shear should be reduced about 20 percent (from Y = 1.15 to 0.95) if only the normal crossing is used to determine the shear force.

Compression Members ($F_{c||}$)

Either the caution or the normal crossing can be critical (i.e., produce the larger compressive force). If the larger of the compressive forces resulting from either the caution or the normal crossings is used, L_n/L'_m is equal to 1.0. If the compressive force is based only on the normal crossing, L_n/L'_m can be as low as 0.80.

A Y value of 1.15 for L_n/L'_m equal to 1.0 appears reasonable and is recommended, based on the following justification. The beta value for L_n/L'_m equal to 1.0 (case 11, Table C1), D_m/L_m equal to 0.1, and Y equal to 1.15 is 2.05 (Table D3). The beta value of 2.05 is 0.54 beta units above the AASHTO illegal overload (case 9) of L_n/L'_m equal to 0.80 and Y equal to 1.0; and 0.33 beta units beneath the AASHTO normal crossing (case 10) of L_n/L'_m equal to 1.0 and Y equal to 1.0.

A Y of 0.95 is recommended for L_n/L'_m equal to 0.80 (case 12, Table C1) for similar reasons as for Y equal to 1.15 and L_n/L'_m equal to 1.0. Again, note the 20 percent reduction (Y = 1.15 to Y = 0.95) in allowable stress required if only the normal crossing is used to determine the compressive member force.

Tension Members (F_t)

Similar arguments can be made for tension members as for compression members. For Douglas fir grade L3, a Y value of 0.90 is reasonable for L_n/L'_m equal to 1.0, and a Y value of 0.75 is reasonable for L_n/L'_m equal to 0.80. The beta value for L_n/L'_m equal to 1.0 (case 17, Table C1), D_m/L_m equal to 0.1, and Y equal to 0.90 is 1.91 (Table D4).

The beta of 1.91 is 0.27 beta units above the AASHTO illegal overload (case 15) of L_n/L'_m equal to 0.80 and Y equal to 0.75; and 0.40 beta units beneath the AASHTO normal crossing (case 16) of L_n/L'_m equal to 1.0 and Y equal to 0.75.

Recent studies and developments* indicate that the current (AITC 117-74) allowable stresses in tension for permanent structures need revision; a proposed set of allowable tensile stresses for industry is being considered. The average ratio of the proposed industrial allowable stress to the current (AITC 117-74) tensile stress level is about 0.67. This is lower than the 0.75 figure used in Table C1 (see note 1), which corresponds to the L3 grade of Douglas fir (Table B2) used in the analysis. As a result, a reduced value of Y of 0.80 ($= \frac{0.67}{0.75} \times 0.90$) is recommended for all grades in tension for the case of L_n/L'_m equal to 1.00, and Y equal to 0.65 ($\approx \frac{0.67}{0.75} \times 0.75$) is recommended for all grades in tension for L_n/L'_m equal to 0.80. Again note the reduction (Y = 0.80 to 0.65) in allowable stress if only the normal crossing force is used.

Summary of the Recommended Allowable Stresses for Temporary Bridges

Table D5 summarizes the recommended allowable stresses and design procedures.

* See tension discussion in Appendix B.

Table D5

Summary of Recommended Allowable Stresses and Design Procedures for Temporary Glued-Laminated Timber Bridge Members

Member (Resistance) Type	Y*	Additional Requirements
Bending (F_b)	1.35	Use recommended lateral load distribution formulas (effective number of stringers) in Appendix E. Use moment corresponding to a normal military crossing in Appendix D, TM 5-312**
Shear (F_v)	1.15	Use recommended procedure for glued-laminated timber in Appendix E to determine shear force
Compression parallel to grain ($F_{c\|\|}$)		
For fully braced compression members (buckling prevented)	1.15	Use larger compressive force resulting from the normal or caution military crossings
	0.95	Compressive force determination based on normal military crossing only
For compression members subject to buckling	1.00	Use larger compressive force resulting from the normal or caution military crossings
	0.80	Compressive force determination based on normal military crossing only
Compression perpendicular to grain ($F_{c\perp}$)	1.15	Use larger compressive force resulting from the normal or caution military crossings
	0.95	Compressive force determination based on normal military crossing only
Tension parallel to grain (F_t)	0.80	Use larger tensile force resulting from normal or caution military crossings
	0.65	Tensile force determination based on normal military crossing only

* Ratio of recommended allowable stress for temporary bridges to that of the AITC 117-74 allowable stress. That is, the allowable stress for temporary bridges, f, becomes: f = YF where Y is given above and F is the AITC 117-74 allowable stress for 10 years of continuous or cumulative loading; F shall include all NDS modifications of stresses (except load duration) including moisture content, size effect (bending), lateral stability (bending), buckling (compression members), and bearing.

** *Military Fixed Bridges*, TM 5-312 (Department of the Army, 1968), with changes 1 and 2.

APPENDIX E:

DEVELOPMENT OF MOMENT DISTRIBUTION
AND SHEAR FORMULAS FOR MILITARY
BRIDGES USING SOLID-SAWN AND
GLUED-LAMINATED TIMBER STRINGERS

The distribution of wheel or track loads is a critical factor in designing the floor systems in military highway bridges. This appendix evaluates the criteria for determining this distribution for both moment and shear in the stringers of timber stringer military bridges and suggests changes or new criteria to improve their validity.

Currently, the criteria for distribution for both military (TM 5-312) and civilian (AASHTO) timber bridges are based primarily on research and experience on bridges with solid-sawn timber stringers with nailed-laminated decks. Although use of glued-laminated timber stringers and deck sections has increased recently, research on the distribution of loads in bridges with these units is very limited. Thus this study was conducted to consider glued-laminated construction.

This appendix discusses the distribution of loads for moment and shear separately. In each case, the basis for the current military criteria is discussed, current research evaluating load distribution indicated, new approaches to distribution outlined, and finally, the validity of these criteria discussed with changes recommended where appropriate.

In general, the approach was the same as that used in evaluating load distribution for moment and shear in steel stringer military bridges (Appendices H and I of *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II). Additional details on the approach are available in that report.

This study was limited to an evaluation of load distribution using currently available methods of analysis and field test data. As new research results--particularly on glued-laminated systems--become available, the recommendations should be reevaluated.

Bridge Configurations

The first step in this study was a review of the types of timber bridges currently being used by the military. The only reference for this review was the 1969 edition of TM 5-302.[9] Since these data were extremely limited, civilian bridges were studied to obtain the basic information on possible types of timber bridges. A broad range of standard bridges developed by the American Institute of Timber Construction,[10] the Canadian Institute of Timber Construction,[11] and the Bureau of Public Roads,[12] as well as selected bridges from the U.S. Forest Service[13] were studied to obtain an indication of the range of design parameters which can be expected in timber military bridges. Table E1 summarizes the bridges reviewed.

Distribution for Moment

The primary thrust of the study of distribution of live load for moment design was related to bridges with glued-laminated stringers; however, limited evaluation indicates that the results are also applicable to closely spaced solid-sawn stringers.

Previous studies[14] have shown that the distribution of wheel loads for moment in the beams of a solid deck bridge can be evaluated

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[9] *Construction in the Theater of Operations*, TM 5-302 (Department of the Army, 1969).

[10] *Glulam Bridge Systems--Plans and Details* (American Institute of Timber Construction, 1974).

[11] *Modern Timber Bridges--Some Standards and Details*, 3rd ed. (Canadian Institute of Timber Construction, Ottawa, Canada, 1970).

[12] *Standard Plan for Highway Bridges*, Volume III, Timber Bridges (U.S. Department of Transportation, Federal Highway Administration, 1969).

[13] Miscellaneous Bridge Plans (U.S. Forest Service, Engineering Division).

[14] W. W. Sanders and H. A. Elleby, *Distribution of Wheel Loads on Highway Bridges*, Report No. 83 (National Cooperative Highway Research Program, 1970); and L. I. Knab et al., *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II, Technical Report M-195 (CERL, 1977), Appendices H and I.

Table E1

Range of Θ for Typical Timber Bridges

Type of Stringer	Width of Roadway W_R[a] (ft)	Span of Bridge L (ft)	Loading	Relative Flexural Stiffness Parameter Θ[b]
Solid-sawn[c]	14	12-49	HS20	0.36-0.97[d]
	20	15-29	HS20	0.82-1.18
	24	13-39	HS20	0.68-1.59
	26	20-36	HS20	0.68-1.10
	30	15-29	HS20	1.20-1.71
	24	11-21	H20	1.53-2.24
	14	13-49	HS30	0.40-1.09
	24	13-39	HS30	0.68-1.89
	14[e]	14-18	Class 50-100	1.47-2.46
	24	14-18	Class 25-60	1.31-2.54
Glued-laminated[f]	26	20-80	HS15	0.69-1.29
	34	20-80	HS15	0.89-1.71
	26	20-80	HS20	0.58-1.04
	34	20-80	HS20	0.75-1.32
	25.2	25-65	H15	0.52-0.81
	25	25-65	H20	0.56-0.91
	14	27-49	HS20	0.31-0.50
	24	39-49	HS20	0.71-0.83
	28	49	HS20	0.78
	34	49	HS20	0.94
	14	13-39	HS30	0.48-0.80
	14	20-55	U90	0.51-0.83

a $W = W_R + 2$

b Θ is defined in Eq E1; development of Θ is discussed in W. W. Sanders and H. A. Elleby, *Distribution of Wheel Loads on Highway Bridges*, Report No. 83 (National Cooperative Highway Research Program, 1970).

c Stringer spacing ranged from 1.31 ft to 2.92 ft (typically about 2.0 ft).

d High value for shorter spans.

e Deck is 3-in. plank (all others with solid-sawn stringers are nailed-laminated).

f Stringer spacing ranged from 4.0 ft to 7.0 ft (typically between 6.0 and 7.0 ft). Decks are either nailed-laminated or glued-laminated with thicknesses from 5 in. to 8 in.

using the orthotropic plate theory. The distribution is related to a relative flexural stiffness parameter Θ and a relative torsional stiffness parameter α. Since α is usually very low and relatively constant (or a low value is assumed, which results in a conservative result), primary consideration was given to a study of Θ:

$$\Theta = \frac{W}{2L}\sqrt[4]{\frac{D_x}{D_y}} \qquad \text{[Eq E1]}$$

where W = width of bridge floor (out-to-out)

L = span of bridge

W/L = aspect ratio

$D_x = E_x I_x$, flexural rigidity per unit width in x direction

$D_y = E_y I_y$, flexural rigidity per unit width in y direction

I_x, I_y = moments of inertia in the x and y directions

E_x, E_y = moduli of elasticity in the x and y directions.

A detailed review of the available bridge types (Table E1) showed that except for a few bridges, the value of Θ ranged between 0.25 and 1.25. Exceptions are generally bridges with aspect ratios greater than 1.0, for which Θ may approach 2.0. A study of Table E1 shows that, in general, for glued-laminated or nailed-laminated deck bridges

single-lane bridges: Θ = 0.30-0.85

double-lane bridges: Θ = 0.50-1.25

Although the decks varied in type (nailed-laminated, plank, multiple-layered, and glued-laminated), the deck was assumed to be solid (i.e., glued-laminated) for the purposes of the initial study. The effect of the actual type of construction is discussed later.

Using the analytical procedures developed for the NCHRP study and following the approach used in the study of steel stringer bridges, a broad spectrum of bridges within the ranges of Θ was evaluated. As noted above, the NCHRP research indicated the validity of

the procedure for steel stringer bridges; however, before the procedures were used for timber bridges, the validity was checked with results of available field tests of several timber bridges.[15] The results of this comparison indicate that these procedures are applicable to solid-deck timber bridges if an average α value of 0.16 (torsional constant) is used.

The major analyses were made for class 60 vehicles, although a limited study was made for class 80 to 90 vehicles (Table E2). Each single-lane bridge was analyzed for the vehicle being placed fully eccentric on the roadway (E1) and for the vehicle being centered on the roadway (C1) (hereafter referred to as the "caution crossing position case"). Each double-lane bridge was analyzed for three conditions: one lane eccentric (E1), two lanes centered (C2), and one lane centered (C1) (Figure E1). Two lanes eccentric is not possible for minimum-width bridges, as the two vehicles essentially cover the entire bridge.

Table E2 shows that:

1. The smaller the Θ value, the better the distribution is (i.e., the effective number of stringers N_E is larger)

2. Larger class vehicles result in better distribution because of the larger track width

3. The critical loading case for single-lane bridges is with the vehicle eccentric (E1)

4. The critical loading case for double-lane bridges is two vehicles centered (C2), with one vehicle eccentric (E1) being the next most critical. The single vehicle centered (C1), which is equivalent

[15] C. Y. Hale, *Field Test of a 40-ft Span Two-Lane Weyerhauser Panelized Wood Bridge*, Report No. RDR-045-1092 (Weyerhauser Co., 1975); *Load Distribution of Stringer Bridges*, Report on Project 8-67-01-400 (U.S. Army Research and Development Laboratories, 1967); and E.C.O. Erickson and K. M. Romstad, *Distribution of Wheel Loads on Timber Bridges*, Research Paper FPL 44 (U.S. Forest Service, 1965).

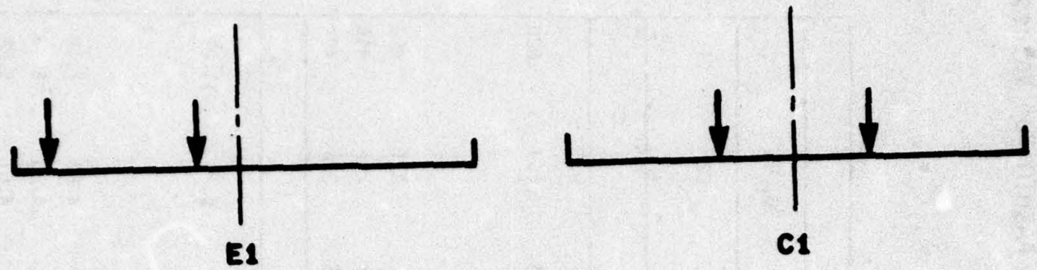
Table E2

Summary of Theoretical Distributions--N_E/N_S and Equivalent AASHTO "D" Values

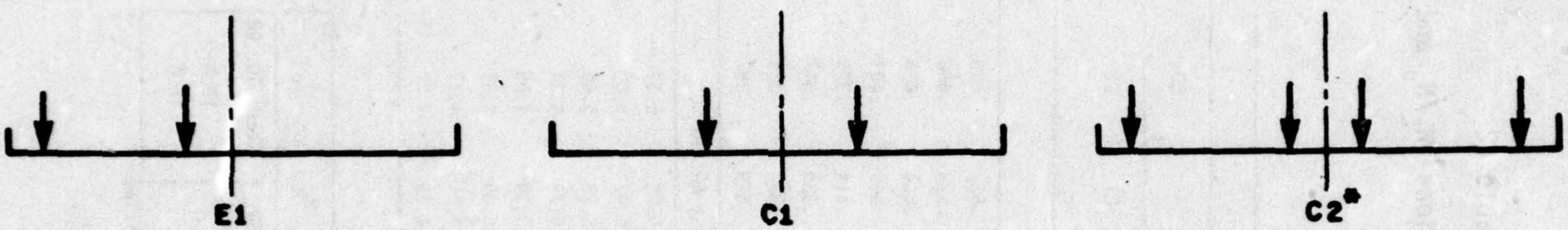
	Single-Lane				Double-Lane					
Bridge Class	60		80, 90		60			80, 90		
θ \ Load case	E1	C1	E1	C1	E1	C2	C1	E1	C2	C1
N_E/N_S										
0.25	.895	.959			.622	.490	.927			
0.50	.772	.842	.779	.870	.528	.466	.803	.508	.463	.820
0.75	.672	.726			.484	.436	.675			
1.00	.587	.622			.445	.403	.584			
1.25	.509	.530			.414	.375	.525			
1.50	.439	.453	.453	.536	.385	.347	.482	.412	.331	.493
1.75	.381	.391	.395	.473	.357	.329	.446	.389	.297	.446
2.00	.335	.344	.349	.423	.329	.297	.414	.365	.276	.404
Equivalent AASHTO "D"										
0.25	6.94	7.43			8.09	6.37	12.05			
0.50	5.98	6.53	6.62	7.40	6.87	6.06	10.44	7.36	6.71	11.89
0.75	5.21	5.62			6.29	5.66	8.77			
1.00	4.55	4.82			5.78	5.24	7.59			
1.25	3.95	4.11			5.39	4.88	6.82			
1.50	3.40	3.51	3.85	4.55	5.00	4.51	6.27	5.98	4.80	7.15
1.75	2.95	3.03	3.36	4.02	4.64	4.17	5.81	5.64	4.30	6.47
2.00	2.60	2.66	2.97	3.60	4.28	3.87	5.39	5.29	4.01	5.85

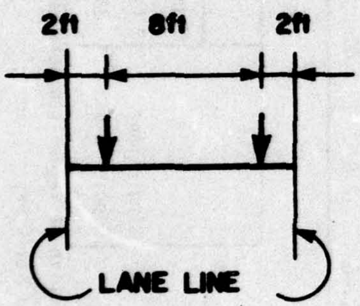
	W_R	
	Class 60	Class 80, 90
Single-lane	13.5	15.0
Double-lane	24.0	27.0

N_E = number of effective stringers (to carry vehicle)
N_S = total number of stringers
$W = W_R + 2$



a. Single lane (W_R = 13.5 ft)





b. Double lane (W_R = 24 ft)

*C2 same as E2 at W_R = 24 ft
(minimum: TM 5-312)

Figure E1. Position of vehicles on roadway (class 60 vehicle).

to the caution crossing position case,* results in the lowest stresses (highest N_E).

Discussion of Results

Table E3 defines the load ratio and shows the effect of Θ and load position on it. The results in Table E3 show that:

1. For a single-lane bridge, the C1 case (caution crossing position) will permit about an 8 percent higher load than the eccentric case (E1), which controlled the development of the current TM 5-312 military criteria and the recommended criteria for steel stringer single-lane bridges.[16]

2. For a double-lane bridge, the two lanes centered case (C2) is critical, with the single lane centered (C1) case, which simulates the caution crossing position, permitting a 40 to 90 percent higher load (C1/C2). If a single vehicle is placed in the most critical position (E1), a 27 to 49 percent higher load can still be permitted without changing the maximum stress.

Table E4 shows the relationships of the current military load distribution criteria in TM 5-312 to those specified for civilian bridges (AASHTO) and to theoretical distribution factors developed previously.

Table E4 also shows the effective number of stringers N_E for the TM 5-312 and AASHTO specifications along with those for the various cases studied. The comparison, which is shown for an expected range of stringer spacing, indicates that the current military criteria for single-lane bridges are more conservative than the AASHTO criteria and substantially more conservative than predicted from the theory. For

* Relates to lateral position; does not include 25 percent increase in vehicle class for caution crossing loading.

[16] L. I. Knab et al., *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II, Technical Report M-195 (CERL, 1977), Appendices H and I.

Table E3

Effect of Number of Vehicles and Vehicle Position (Class 60)

Θ	Load Ratio*			
	Single-Lane Bridge	Double-Lane Bridge		
	C1/E1*	C1/E1*	E1/C2*	C1/C2*
0.25	1.07	1.49	1.27	1.89
0.50	1.09	1.52	1.13	1.72
0.75	1.08	1.39	1.11	1.55
1.00	1.06	1.31	1.10	1.45
1.25	1.04	1.27	1.10	1.40
1.50	1.03	1.25	1.11	1.39
1.75	1.03	1.25	1.11	1.39
2.00	1.03	1.26	1.11	1.39

* The load ratio is the ratio of load that can be carried by vehicle in position indicated by numerator of ratio to that with vehicle in position indicated by denominator of ratio.

Table E4a

Relationship of Specification Capacity and Theory Capacity (Class 60) at Minimum Width for Single-Lane Glued-Laminated Timber Bridges ($\Theta = 0.25$ to 0.75, $\alpha = 0.16$)--Capacity in Terms of Effective Number of Stringers

Basis for N	Wide S_s	Narrow S_s
Stringer spacing	$S_s = 4.50$ ft	$S_s = 2.70$ ft
Number of stringers	$N_s = 4$	$N_s = 6$
Military N_1	2.11	2.85
AASHTO*	2.22	3.70
Theory (EI)		
Full range - Θ	2.69-3.58	4.03-5.37
Most common - Θ**	3.09	4.63
Theory (CI)		
Full range - Θ	2.90-3.84	4.36-5.75
Most common - Θ	3.37	5.05

* D = 5.0 for timber deck (strip 6 in. or more thick), interior stringer.

** Most common $\Theta = 0.50$ for single-lane bridges.

Table E4b

Relationship of Specification Capacity and Theory Capacity (Class 60) at Minimum Width for Double-Lane Glued-Laminated Timber Bridges (Θ = 0.25 to 1.25, α = 0.16)--Capacity in Terms of Effective Number of Stringers

Basis for N	Extra Wide S_s	Wide S_s	Narrow S_s
Stringer spacing	S_s = 8.00 ft	S_s = 6.00 ft	S_s = 2.40 ft
Number of stringers	N_s = 4	N_s = 5	N_s = 11
Military N_1	1.63	1.83	3.08
Military N_2	1.50	1.88*	4.13*
AASHTO (one-lane)**	1.60	2.00	4.17
AASHTO (two-lane)†	1.14	1.42	3.54
Theory (E1)			
Full range - Θ	1.66-2.49	2.07-3.11	4.55-6.84
Most common - Θ††	1.94	2.42	5.32
Theory (C2)			
Full range - Θ	1.50-1.96	1.88-2.45	4.13-5.39
Most common - Θ	1.74	2.18	4.80
Theory (C1)			
Full range - Θ	2.10-3.71	2.63-4.64	5.78-10.20
Most common - Θ	2.70	3.38	7.43

* Does not apply, since $N_2 > N_1$.

** D = 5.0 for timber deck (strip 6 in. or more thick), interior stringer.

† D = 4.25 for timber deck (strip 6 in. or more thick), interior stringer.

†† Most common Θ = 0.75 for double-lane bridges.

double-lane bridges, the military criteria are more conservative than the AASHTO criteria at narrow S_s, but less conservative (compared to theory) at extra wide spacing. In any case, the current military criteria are equally or more conservative than any theory case considered.

Table E5 shows a more realistic comparison of the current military load distribution compared to real behavior and other criteria. This table shows the relationship between N_E (the number of effective stringers) as predicted by theory and that predicted by current military criteria. It should be remembered that the theoretical values assume the deck to be solid (i.e., glued-laminated). Many timber decks (nailed-laminated, plank, etc.) will result in poorer distributions.

Table E5 shows that for single-lane bridges, the caution loading case (defined in Table E5) is critical. Assuming that 125 percent of normal one-way class loading would be permitted in that case, the ratio of $N_{theory}/N_{military}$ (N_Θ/N_M) ranges from 1.10 to 1.61 (i.e., the true class without actually exceeding the design stress is 110 to 161 percent of design class), with it normally expected to range from 1.28 to 1.42. This compares with a range in that ratio from 1.27 to 1.88 for the general loading case (normally from 1.46 to 1.62).

Table E5 also shows that the general case is critical for double-lane bridges. In this case, the ratio ranges from 1.00 to 1.75, with a most likely range of 1.16 to 1.56. For the caution loading case (defined in Table E5), the ratio of N_{theory} to $N_{military}$ ranges from 1.03 to 2.65, with the typical value being between 1.33 and 1.93.

Unusual Cases

When the width of bridges becomes relatively large with respect to the span (i.e., aspect ratio W/L greater than 1), the values of Θ approach the upper part of the range of Θ values considered (Θ = 0.25 to 1.25). In fact, Θ values can be higher than 1.25. A review of Tables E4 and E5 indicates that at these higher values of Θ the

Table E5

Relationship of Number of Effective Stringers From Theory to Current Military Criteria for Glued-Laminated Timber Bridges

Number of Lanes N_L	S_s [a]	Range of N_Θ/N_M [b]		
		General Case[c]	Central Loading[d]	Caution Loading[e]
1	Wide	1.27-1.70 (1.46)[f]	1.37-1.82 (1.60)	1.10-1.46 (1.28)
1	Narrow	1.41-1.88 (1.62)	1.53-2.02 (1.77)	1.22-1.61 (1.42)
≥2	Extra Wide	1.00-1.31 (1.16)	1.29-2.28 (1.66)	1.03-1.82 (1.33)
≥2	Wide	1.03-1.34 (1.19)	1.44-2.54 (1.85)	1.15-2.03 (1.48)
≥2	Narrow	1.34-1.75 (1.56)	1.88-3.31 (2.41)	1.50-2.65 (1.93)

a S_S as defined in Table E4
b Values of N obtained from Table E4, N_θ = number of effective stringers from theory; N_M = number of effective stringers from current military criteria.
c For single-lane bridges, case E1 controls; for double-lane bridges, case C2 controls (Figure E1).
d One lane centered loading (C1) as used for caution crossing position case.
e 125 percent of N_M for caution crossing position case (C1) used to consider 125 percent of one-way class for caution loading.
f Numbers in parentheses are the ratios for most common value of Θ, which is about 0.50 for single-lane bridges and 0.75 for double-lane bridges.

current military criteria for load distribution are reasonably accurate in predicting behavior for the critical loading cases. However, for extreme cases (a short-span, double-lane bridge) with wider stringer spacing, the current military criteria may actually be unconservative compared to theory.

Bridges with large aspect ratios are generally short double-lane bridges. It is not likely that large aspect ratios will occur often in single-lane timber stringer bridges (particularly those with glued-laminated stringers).

Nailed-Laminated Decks

Although the deck was assumed to be solid (i.e., glued-laminated) in the analyses discussed above, it is expected that decks which are not "solid" will be used in theaters of operations in many instances. These decks will generally be nailed-laminated decks, which research has shown to be initially less efficient in distributing load than the "solid" glued-laminated deck.[17] In addition, the nails tend to work loose with time, causing a further reduction in distribution capabilities due to loss of torsional rigidity.

The review of field tests of military bridges with timber decks[18] indicated that an α value (torsional constant) of 0.16 is realistic for a timber deck bridge. This value of α was determined from studies of bridges with narrow stringer spacings. It is expected that wider spacings may be used, particularly for glued-laminated stringers, and would possibly result in lower α values. If this rigidity is reduced to zero (no torsional rigidity), an indication of the effect of loss

[17] E.C.O. Erickson and K. M. Romstad, *Distribution of Wheel Loads on Timber Bridges*, Research Paper FPL 44 (U.S. Forest Service, 1965).
[18] L. I. Knab et al., *Design Criteria for Theater of Operations Steel Highway Bridges*, Volume II, Technical Report M-195 (CERL, 1977), Appendices H and I.

of laminating characteristics can be obtained. Using the results of the NCHRP study, it is estimated that a reduction in N_E of 10 to 15 percent can result from the loss of torsional rigidity. Thus, the average difference between "true class" and that computed by current military criteria (Table E5) will be reduced to about 100 to 130 percent with a typical value of about 110 percent.

Special Studies

Although the general study was directed toward class 60 to 90 bridges, the results should be applicable to both higher and lower class bridges. Although the lighter class vehicles have narrow wheel spacings (track), the Θ values for bridges of these classes (20-40) are lower than expected for classes 60 to 90. The lower values of Θ have the highest conservatism.

For the heavier class bridges (up to class 150), the Θ values are still within the range studied, and the wider track would assist in providing better load distribution. Thus, it is felt this study shows trends which should be applicable for all classes of vehicles.

Summary

The results of the study can be summarized as follows. The results are based on the ranges of variables studied, but it is felt that, within general limits, they will apply to most timber military bridges. The results apply to lateral load distribution criteria and specifications only; other criteria, such as allowable stress, are not considered. Unless otherwise stated, use of "criteria" or "specification" refers to the military requirements in TM 5-312.

1. For most military bridges, the current criteria for the design of beams for flexure (moment) generally underestimate the load-carrying capacity of the bridge.

2. For typical single-lane, solid-deck* bridges, the true moment for the general loading case (Table E5) is at least 20 percent less than predicted by current criteria. The moment for the caution crossing position case** averages about 10 percent higher than that for the general loading case. For the caution crossing position case with a 25 percent increase above the normal crossing class, the true capacity is at least 10 percent higher than predicted by current criteria.

3. For typical double-lane, solid-deck bridges, the true moment capacity for the general case (Table E5) is generally from 15 to 55 percent higher than predicted by specification; however, for wide spacings this percentage can become very low. For typical bridges, the increase for the caution crossing position case with 25 percent increase in class from the general case is at least 15 percent.

4. The current criteria seem to adequately predict the behavior of bridges with high aspect ratios (i.e., $W/L > 1$, where W = width of bridge floor, L = span length). In some cases, however, if W/L is significantly greater than 1, the current criteria can actually become unconservative.

5. The AASHTO load distribution criteria do not appear to provide any significantly better indication of behavior than the current military load distribution criteria. However, both criteria appear to be conservative for most bridges.

6. For typical single-lane, solid-deck bridges, the caution crossing position case with 125 percent of normal one-way crossing is critical. For double-lane, solid-deck bridges, the general loading case (using current criteria for N_1 and N_2) is critical. For the critical case for single-lane, solid-deck bridges the ratio of N by

* Glued-laminated panel or concrete deck.

** Unless otherwise stated, the caution crossing position case relates to lateral position and does not include a 25 percent increase in vehicle class for the caution crossing loading.

theory to N by military specification ranges from 1.10 to 1.61 for the expected spectrum of bridges with an expected value of 1.28 to 1.42. The average value of the ratio is about 1.35. For typical double-lane, solid-deck bridges, the ratio for the critical condition ranges from 1.00 to 1.75 with the expected range from 1.16 to 1.56. For double-lane bridges, a ratio of 1.20 to 1.25 would be about the average value. For both the single- and double-lane cases, a representative average value of N by theory to N by specification is about 1.25.

7. For both single- and double-lane bridges with nailed-laminated, planked, or multiple-layered decks, the ratio of N by theory to N by specification ranges from about 1.00 to 1.35, with an average value of about 1.10 (compared to an average value of 1.25 for solid decks).

8. The conservatism of the current lateral load distribution criteria can be compensated for by increasing the allowable stresses for flexure (moment) as developed in Appendix D. The effects of large W/L ratios and the use of decks which are not solid can be accounted for by reducing the effective number of stringers by a reduction factor c, as given in Table E6b.

Recommendations

Based on the material presented herein, the following recommendations are made:

1. The current equations used to determine the effective number of stringers in Paragraph 6-5 of TM 5-312 should be replaced by the recommended equations given in Tables E6a and E6b. The recommendations for the effective number of stringers should be combined with the allowable stress recommendations given in Appendix D for glued-laminated stringers. Note that the allowable stresses for solid-sawn timber stringers have not been considered in this report.

2. A significant increase in class of vehicle permitted on the

Table E6a

Recommended Equations and Current TM 5-312 Equations for Determining the Effective Number of Stringers

Current TM 5-312 Equations		Recommended Equations
Single lane: $N_1 = \frac{5}{S_s} + 1$	(Eq 6-7a)	Single lane: $N_1 = c[\frac{5}{S_s} + 1]$
Two lanes: $N_2 = \frac{3}{8} N_s$ or $N_2 = N_1$ whichever is smaller	(Eq 6-7b)	Two lanes: $N_2 = c[\frac{3}{8} N_s]$ or $N_2 = N_1$ whichever is smaller

where N_1 = effective number of stringers for single-lane bridges
S_s = center-to center stringer spacing in feet
N_s = number of stringers
N_2 = effective number of stringers for two-lane bridges
c = reduction factor given in Table E6b

Table E6b

Values of Reduction Factor c* Used in Recommended Formulas in Table E6a for Determining the Effective Number of Stringers

Bridge Deck Type	Ratio of Bridge Floor Width (out-to-out) to Bridge Span Length (W/L)	
	$W/L \leq 1.0$	$W/L > 1.0$
Glued-laminated timber or concrete	1.0	0.75
Nailed-laminated timber, plank, or multiple-layered	0.90	0.70

* c accounts for the reduction in lateral load distribution when using nailed-laminated timber, plank, or multiple-layered decks and/or bridges which are very wide compared to their span length.

bridge may result in extremely high deck stresses. The designer should be cautioned to check the deck design.

Distribution for Shear

The current TM 5-312 criteria for live load shear distribution for timber stringer bridges states that

$$v_{LL} = \frac{3V_{LL}}{16}\left(\frac{N_1 \text{ or } N_2 + 1}{N_1 \text{ or } N_2}\right) \qquad \text{[Eq E2]}$$

where v_{LL} = shear per stringer

V_{LL} = undistributed live load shear

N = number of effective stringers for moment distribution.

These criteria were developed based on research conducted on short-span, closely spaced solid-sawn stringers.[19] A study of recent research indicates that the approach is slightly conservative for this bridge type.

On the other hand, the shear distribution of glulam stringer bridges (which normally have larger stringer spacings than solid-sawn stringers) is considerably different. This study indicated that the distribution is similar to that found for steel stringer bridges.

Thus, the distribution of live load for shear requires two criteria, depending on the type of stringer. The recommended criteria and the background of the development are therefore presented separately for solid-sawn and glulam stringers.

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[19] E.C.O. Erickson and K. M. Romstad, *Distribution of Wheel Loads on Timber Bridges*, Research Paper FPL 44 (U.S. Forest Service, 1965); J. A. Newlin, G. E. Hack, and H. W. March, "New Method of Calculating Longitudinal Shear in Checked Wooden Beams," *Transactions of the American Society of Mechanical Engineers* (1934), pp 739-744; and *National Design Specification for Stress Grade Lumber and Its Fastenings* (National Forest Products Association, 1973).

Shear Distribution in Solid-Sawn Stringer Bridges

One of the basic assumptions[20] in the development of the shear distribution criteria in timber bridges for both civilian (AASHTO) and military (TM 5-312) bridges is that all beams are checked. These checks (or splits) cause a change in behavior.

For these timber bridges, the span is usually relatively short. Thus, there is considerable lateral distribution, and the effective live load shear is the average of the distributed shear and the undistributed shear on one stringer (TM 5-312):

$$\frac{V_{LL}}{4}\left(1 + \frac{1}{N_{1,2}}\right) \qquad \text{[Eq E3]}$$

This shear is similar to that required by AASHTO for civilian bridges. Research conducted by the Forest Products Laboratory[21] showed that the current AASHTO procedure is slightly conservative for the typical solid-sawn stringer bridge. The AASHTO criteria (converted to military design terminology) are the same except the term in brackets is

$$\left(0.6 + \frac{2}{N_{1,2}}\right)$$

Since $N_{1,2}$ usually ranges from about 3 to 5 for military bridges, the constant term will be a predominant factor, and the AASHTO criteria will give a lower effective live load shear than that of the military (Table E7).

The stringer shear determined in Eq E3 is based on the loads

[20] Erickson and Romstad; Newlin, Hack, and March; *National Design Specification*; J. A. Newlin, "Shear in Checked Beams," *Proceedings of the American Railway Engineering Association* (1934), pp 1001-1004; and W. D. Keeney, "Some Notes on Highway Timber Trestle Design," *Wood Preserving News* (June 1941), pp 73-79.

[21] Erickson and Romstad.

Table E7

Bracketed Terms in Shear Distribution Criteria for Single-Lane Military and AASHTO Bridges

		Bracketed Factor in Military Criteria	Bracketed Factor in AASHTO Criteria	
S_s	N_1	V_M	V_C	V_M/V_C
1.5	4.3	1.23	1.07	1.15
2.0	3.5	1.29	1.17	1.10
2.5	3.0	1.33	1.27	1.05

being placed to maximize shear at the support. Tests with timber beams[22] indicate that the critical case for shear (due to checking) occurs when the concentrated load is actually some distance from rather than at the support. This change is primarily due to the increase in horizontal shear strength of checked beams due to the compressed fibers for loads near the support.

The research showed that the optimum (or critical) condition exists when the loads are placed at a distance three times the beam depth d, but not farther than one-fourth of the span length L from the support. Since military timber bridges (solid-sawn stringers) tend to have extremely short spans, the critical condition will normally occur near the L/4 distance from the support. As a result, the current military criteria simply allow the designer to use 3/4 of the shear calculated from Eq E3 as the design live load shear per stringer. In effect, this changes the factor $V_{LL}/4$ to $3V_{LL}/16$.

[22] E.C.O. Erickson and K. M. Romstad, *Distribution of Wheel Loads on Timber Bridges*, Research Paper FPL 44 (U.S. Forest Service, 1965); J. A. Newlin, "Shear in Checked Beams," *Proceedings of the American Railway Engineering Association* (February 1934), pp 1001-1004.

Use of 3/4 of the calculated shear tends to increase the conservatism of the military criteria. If, in fact, the loads are moved away from the reaction a distance of L/4, for a short span, some of the loads will move off the span, resulting in a greater reduction in shear than that calculated using 3/4 of the reaction shear. This conservatism could be reduced if there are shallow beams for which the distance to the load should be 3d rather than L/4.

As noted previously, earlier research has shown that the current AASHTO criteria are reasonable and appear to predict the critical horizontal shear in timber stringer bridges fairly accurately. Thus, the military criteria are somewhat conservative, with the degree of conservatism depending on the stringer spacing and the span. It can be seen from Table E7 that for a typical stringer spacing of 24 in., the military criteria estimate shear 10 percent higher than AASHTO. This difference could be even greater for a short span, where use of a straight reduction in reaction shear does not consider the loads moving off the span (by moving the axle nearest reaction to quarter-span).

In summary, it is estimated that the military criteria overestimate the actual shear (assuming current AASHTO criteria to be satisfactory) by about 5 to 15 percent. It should be remembered, however, that the entire development of both the AASHTO and military criteria are based on checked timber stringers and, thus, the allowable horizontal shear stress should reflect this condition.

Shear Distribution in Glued-Laminated Stringer Bridges

As noted earlier, the research on glued-laminated stringer bridges is extremely limited. At present, there appears to be no research information available on distribution of live load for shear. The current load distribution criteria for military and AASHTO glued-laminated bridges are still the same as those for bridges with solid-sawn stringers.

This approach does not seem to be realistic, as the current distribution criteria are based on closely spaced checked stringers in short-span bridges. In glued-laminated bridges, the spacing (as seen in Table E1) ranges from 4 to 7 ft, with spans up to 80 ft. These ranges are markedly different from those seen in solid-sawn stringer bridges, but the major difference is in the assumption of a "checked or split" beam in the development of the solid-sawn stringer shear. The glued-laminated stringer is essentially unchecked and thus behaves differently.

It appears from a study of the development of the shear distribution criteria for both steel and timber stringer bridges as specified by TM 5-312 and AASHTO that the approach used for steel stringer bridges would more accurately reflect the true behavior of glued-laminated stringer bridges. In *Design Criteria for Theater of Operations Steel Highway Bridges*, it is recommended that the military change its shear distribution procedure for steel stringers to the approach used by AASHTO. Thus, using this same approach for glued-laminated military bridges appears appropriate. The development of these criteria is detailed in the above-referenced report; a brief summary is presented here.

The current Army steel stringer criteria (TM 5-312) simply state that one-half of the shear from a single vehicle shall be carried by each stringer. However, for glued-laminated and steel stringers, the distribution of shear depends significantly on the longitudinal, as well as the transverse, placement of loads on the bridge deck surface. The longitudinal placement of vehicles on the bridge to maximize shear results in the vehicle being placed near the reaction.

The transverse distribution of these loads is affected by the flexibility of the entire floor as well as the transverse placement of the loads. As the loads are moved longitudinally away from the reactions, the floor tends to deform more, resulting in distributions which conform more to that which is used for moment.

Since the beams do not deflect at the reactions, the loads are distributed laterally by the deck behaving as if it was a series of simple beams supported by the stringers. This is based on the assumption that transverse continuity of the deck is minimal at the stringers. This approach to shear distribution is the same as that used in the current AASHTO specification. Basically, the procedure is to calculate the shear per stringer for wheels or tracks near the reaction, assuming that the deck acts as a series of simple beams. The loads out on the span are distributed in the same manner used for moment.

For tracked vehicles, the distribution of shear is simple, since all of the load is concentrated over a short longitudinal distance and has one track configuration per class. Thus, since the load is near the reaction, the deck does not deflect significantly and the load can be distributed transversely, assuming that the deck acts as a series of simple beams (Figure E2).

The shear distribution for wheeled vehicles is different than for tracked vehicles, since many of the wheels are longitudinally placed significantly farther out on the span. Thus, the deck has the opportunity to deform, and it can be assumed that the transverse distribution of load for shear conforms substantially to that for moment (in this case, the wheels are assumed to be near midspan). This is a conservative assumption which will be adequate for design.

Thus, the suggested design criteria are based on a simple beam distribution for the axle near the reaction (Figure E3) and on the distribution criteria for moment for axles out on the span proper. The determination of the appropriate loads to be distributed can be obtained by using Appendix D of TM 5-312. The shear given in that appendix is for the entire vehicle, and the reaction shear (due to axle over the reaction) is simply the weight of the heaviest axle* with the shear to be "distributed" equaling the remainder.

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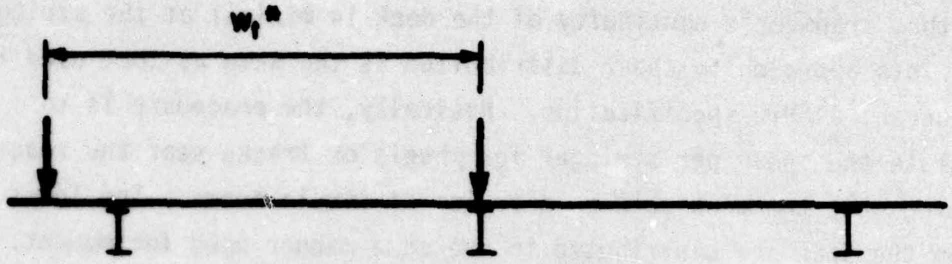
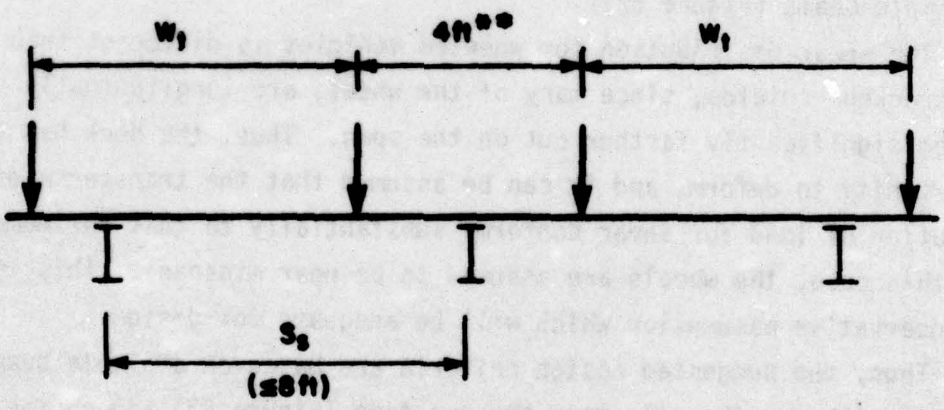
* For classes 40 and 50, the critical shear does not occur with the heaviest axle at the reaction; however, using the heaviest axle as the axle at the reaction will result in conservative estimates of the critical shear.



a. Single vehicle.

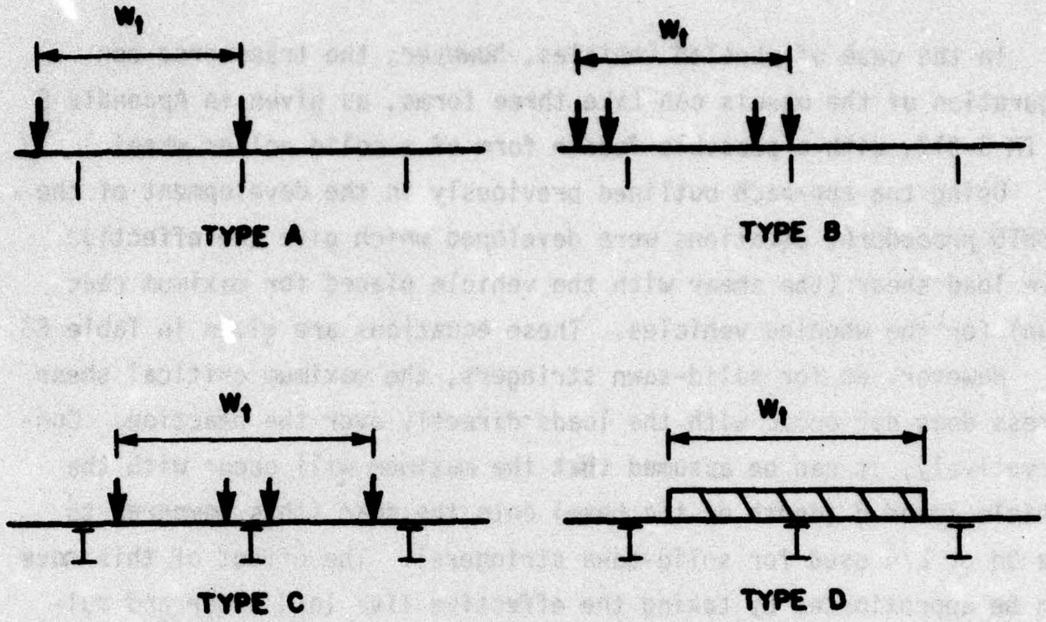


b. Multiple vehicles.

* W_t = track or wheel spacing, i.e., center-to-center distance between tracks.

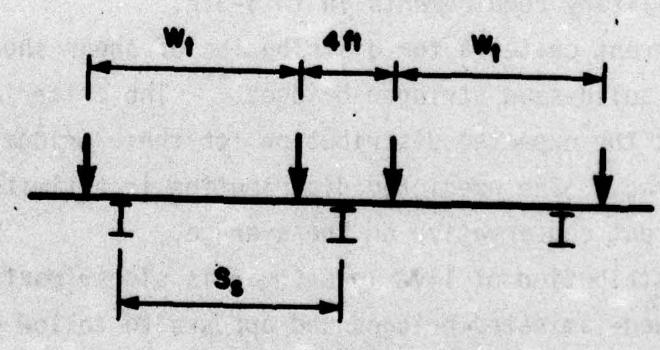
** Minimum distance between tracks assumed to be 4 ft.

Figure E2. Transverse load positions for tracked vehicles for reaction shear.



Types A through C refer to columns 5 through 7 of Appendix D-1, TM 5-312.

a. Single vehicle.



b. Multiple vehicles.

Figure E3. Transverse load positions for wheeled vehicles for reaction shear.

In the case of wheeled vehicles, however, the transverse configuration of the wheels can take three forms, as given in Appendix D of TM 5-312, with a possible fourth form of a solid roller wheel.

Using the approach outlined previously in the development of the AASHTO procedure, equations were developed which give the effective live load shear (the shear with the vehicle placed for maximum reaction) for the wheeled vehicles. These equations are given in Table E8.

However, as for solid-sawn stringers, the maximum critical shear stress does not occur with the loads directly over the reaction. Conservatively, it can be assumed that the maximum will occur with the vehicle moved d (depth of the beam) onto the span (this compares to the 3d or L/4 used for solid-sawn stringers). The effect of this move can be approximated by taking the effective live load shear and multiplying it by (L-d)/L. Since most of the loads will stay on the span, this is a reasonable approximation.

Summary

The following results apply to shear distribution criteria and specifications only; other criteria, such as allowable shear stress, are not considered. Unless otherwise stated, use of "criteria" refers to the military requirements in TM 5-312.

1. The current criteria for distribution of shear should be applied only to solid-sawn stringer bridges.** The criteria do, however, predict the expected distribution for these bridges with reasonable accuracy. The predicted distribution is estimated to be about 10 percent conservative on the average.

2. The distribution of live load shear is significantly different in glued-laminated bridges and appears to follow closely the criteria used by AASHTO for steel or concrete bridges. The

** Criteria should be modified for the increase in class for the caution crossing loading--see *Recommendations* in this section.

Table E8

Value of Effective Live Load Shear Force per Stringer, v'_{LL}

	v_{LL}' for Single Lane,* kips	v_{LL}' for Double Lane,** kips
Wheeled vehicle	$1.25\left[\left(0.5 + \frac{S_s}{32}\right)V_A + \left(\frac{V_{LL_W} - V_A}{N_1}\right)\right]$	$\left(\frac{S_s - 2}{S_s}\right)V_A + \left(\frac{V_{LL_W} - V_A}{N_2}\right)$
Tracked vehicle	$1.25\left(\frac{V_{LL_T}}{2}\right)$	$\left(\frac{S_s - 2}{S_s}\right)V_{LL_T}$

where: V_{LL_W} = wheeled vehicle shear in kips as given in Appendices D-4 and D-7† of TM 5-312

V_{LL_T} = tracked vehicle shear in kips as given in Appendices D-5 and D-7† of TM 5-312

V_A = the heaviest axle load in kips as given in column 3† of Appendix D-1 of TM 5-312

S_s = center-to-center stringer spacing in ft

N_1 = effective number of stringers for single lane given in Table E6a.

N_2 = effective number of stringers for double lane given in Table E6a.

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* The coefficient of 1.25 is used to adjust shear from normal crossing case to caution crossing case.

** For the double-lane case, v'_{LL} shall be computed for both single and double lanes and the larger value of v_{LL} shall be used.

† Entries in Appendix D-7 and column 3 of Appendix D-1 are given in tons and must be converted to kips.

AASHTO procedure was used to develop the criteria for live load shear distribution in steel stringer military bridges and to determine the effective live load shear given in Table E8 for glued-laminated timber stringer military bridges.

3. The critical shear stress for both stringer types does not occur with the vehicle at the position for maximum reaction (at the support). In each case, the critical shear stress occurs with the vehicle out on the span. For solid-sawn stringers, the vehicle should be moved from the reaction onto the span a distance three times the beam depth (but not farther than one fourth of the span length); for glued-laminated stringers, it should be moved out a distance equal to the stringer depth. This movement results from the loads at the reaction causing significant increases in the horizontal shear strength.

Recommendations

The following recommendations are made for determining the distribution of live load shear in military timber stringer bridges:

1. For glued-laminated timber stringers, the following recommended procedure should be incorporated into Paragraph 6-6 of TM 5-312; the procedure should be used with the allowable shear stress recommended in Appendix D.

6-6. Shear Check (Shear Design)

c. *Glued-Laminated Timber Stringer Bridges*

Compared to steel stringers, glued-laminated timber stringers are relatively lower in horizontal shear strength. Thus, shear can be critical in glued-laminated timber stringers and the horizontal shear stress often controls the allowable design load, particularly for short spans.

(1) *Dead Load Shear*. The dead load shear per stringer is determined as for a steel stringer bridge using

$$v_{DL} = \frac{V_{DL}}{N_s} \qquad \text{(Equation 6-12)}$$

(2) *Live Load Shear.*

(a) *Effective shear per stringer,* v'_{LL}*, for glued-laminated timber stringers.* The effective live load shear per stringer, v'_{LL}, must account for the shear per stringer due to loads near the support as well as loads out on the span. It can be assumed that the wheel or track loads which are at or near a support will go directly into the stringer (with the deck assumed to act as a series of simple beams) and that the loads which are out on the span will be distributed laterally in a manner similar to moment. The effective live load shear per stringer v'_{LL} in kips shall be determined from Table E8.

(b) *Design live load shear per stringer,* v_{LL}*, for glued-laminated timber stringers.* Tests with timber beams indicate that the shear failure will occur when a concentrated load is at some constant distance from the support, rather than when the load is just off the support, the location that produces maximum shear. This is caused by the concentrated load tending to compress the fibers, thus increasing the horizontal shear strength. When the load is moved off the support a distance of about the depth of the stringer, the optimum condition for shear failure exists. Thus, the value of the effective live load shear per stringer, v'_{LL}, should be reduced accordingly. The design live load shear per stringer, v_{LL}, in kips, is:

$$v_{LL} = \left(\frac{L - d}{L}\right)v'_{LL}, \text{ but not less than } 0.75\ v'_{LL}$$

(Revised Equation 6-17)

where v_{LL} = design live load shear per stringer in kips
L = bridge span in ft
d = depth of stringer in ft
v'_{LL} = effective live load shear per stringer from Table E8.

(3) *Total Shear Per Stringer.* The total design shear, v, for a glued-laminated timber stringer is:

$$v = v_{DL} + v_{LL}$$

where v = total design shear per stringer in kips
v_{DL} = dead load shear per stringer in kips
v_{LL} = design live load shear per stringer in kips.

The rest of the shear design procedure shall be the same as that given in subparagraphs (4) and (5) of paragraph 6-6b, *Timber Stringer Bridges*, of TM 5-312.

2. For solid-sawn timber stringers, the following procedure is recommended. Note that the allowable shear stresses for solid-sawn timber stringers have not been considered in this report.

6-6. Shear Check (Shear Design)*

b. *Solid-Sawn Timber Stringer Bridges* (The introductory paragraph in this subparagraph is not modified.)

(1) *Dead Load Shear* (This subparagraph is not modified.)

* Refers to paragraph 6-6 of TM 5-312.

(2) *Live Load Shear.*

(a) *Effective Shear,* v'_{LL} *for solid-sawn stringers*

Since the timber stringer bridge has a relatively short span in relation to the design vehicle, considerable lateral distribution of the loads will take place. It can be assumed that the wheel or track loads which are at or near a support will go directly into the stringer (with the deck assumed to act as a series of simple beams) and the loads which are out on the span will be distributed laterally in a manner similar to moment. Thus, the effective live load shear per stringer, v'_{LL}, is taken as the average of the undistributed and distributed shear for one track or wheel live load.*

For single lane, v'_{LL} is:

$$v'_{LL} = 1.25 \left(\frac{V_{LL}}{4}\right) \left[\frac{N_1 + 1}{N_1}\right] \quad \text{(Revised Equation 6-16a)}$$

(The value of 1.25 is used to adjust shear from the normal crossing load case to shear for the caution crossing load case.)

and for double lane, v'_{LL} is:

$$v'_{LL} = \left(\frac{V_{LL}}{4}\right) \left[\frac{N_2 + 1}{N_2}\right] \text{ or } v'_{LL} = 1.25 \left(\frac{V_{LL}}{4}\right) \left[\frac{N_1 + 1}{N_1}\right] \quad \text{(Revised Equation 6-16b)}$$

whichever is larger

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* In general, v'_{LL} can be written as:

$$v'_{LL} = K\left[\frac{V_{LL}}{4} + \left[\frac{V_{LL}}{4}\left(\frac{1}{N_1 \text{ or } N_2}\right)\right]\right] = K\left(\frac{V_{LL}}{4}\right)\left[\frac{(N_1 \text{ or } N_2) + 1}{N_1 \text{ or } N_2}\right]$$

where K is either 1.25 or 1.0. See revised Equation 6-16a and b for recommended v'_{LL} formulas for single and double lanes.

where v'_{LL} = effective live load shear per stringer, in kips

V_{LL} = undistributed live load shear in kips from Appendix D of TM 5-312

N_1 = effective number of stringers for single lane given in Table E6a

N_2 = effective number of stringers for double lane given in Table E6a.

(b) *Design live load shear, v_{LL}, for solid-sawn stringers.*

Since timber is weak in horizontal shear, the horizontal shear stress controls the allowable shear load. However, the values of live load shear are reduced from the theoretical value determined from revised Equations 16a and b. Tests with timber beams indicate that the shear failure will occur when a concentrated load is at some constant distance from the support, rather than when the load is just off the support, the location that produces maximum shear. This is caused by the concentrated load tending to compress the timber fibers, thus increasing the horizontal shear strength. When the load is moved off the support a distance of about 3 times the stringer depth or 1/4 the span length, the optimum condition for shear failure exists. The live load design shear, v_{LL}, is then taken as

$$v_{LL} = \left(\frac{L - 3d}{L}\right) v'_{LL}\text{, but not less than } 0.75\, v'_{LL} \qquad \text{(Revised Equation 6-17)}$$

where v_{LL} = live load design shear per stringer in kips

L = bridge span in ft

d = stringer depth in ft

v'_{LL} = effective live load shear per stringer in kips, from revised Equations 6-16a and b.

(3) *Total shear per stringer, for solid-sawn stringers.* The total design shear v for a solid-sawn timber stringer is

$$v = v_{DL} + v_{LL} \qquad \text{(Revised Equation 6-18)}$$

where v = total design shear per stringer in kips

v_{DL} = dead load shear per stringer in kips, given by Equation 6-12

v_{LL} = live load design shear per stringer in kips, given by revised Equation 6-17. The rest of the shear design procedure is the same as that given in subparagraphs (4) and (5) of paragraph 6-6b, *Timber Stringer Bridges*, in TM 5-312.

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SI CONVERSION FACTORS

1 ft	= 0.3048 m
1 ft-lb	= 1.3558 Nm
1 in.	= 2.54 cm
1 kip	= 4.448 kN
1 kip-ft	= 1.3558 kNm
1 ksi	= 0.69 kN/cm^2
1 sq in.	= 6.4516 cm^2
(°F -32) 5/9	= °C

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